CHAPTER 13

STORM DRAINAGE SYSTEMS

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13.1 OVERVIEW

13.1.1 Introduction

This Chapter provides guidance on storm drain design and analysis. The quality of the final in-place system usually reflects the attention given to every aspect of the design and that accorded to the construction and maintenance of the facility. Suggested policy and guidelines are given to encourage user Agencies to develop their own policies and design criteria. Most aspects of storm drain design (e.g., system planning, pavement drainage, gutter flow calculations, inlet spacing, pipe sizing, hydraulic grade line calculations) are included.

The design of a drainage system must address the needs of the traveling public and those of the local community through which it passes. The drainage system for a roadway traversing an urbanized region is more complex than for roadways traversing sparsely settled rural areas. This is often due to:

- the wide roadway sections, flat grades (both in longitudinal and transverse directions), shallow water courses and absence of side channels;
- the more costly property damages that may occur from ponding of water or from flow of water through builtup areas; and
- the fact that the roadway section must carry traffic but also act as a channel to convey the water to a disposal point. Unless proper precautions are taken, this flow of water along the roadway will interfere with or possibly halt the passage of highway traffic.

13.1.2 Inadequate Drainage

The most serious effects of an inadequate roadway drainage system are:

- damage to surrounding or adjacent property, resulting from water overflowing the roadway curbs and entering such property;
- risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway; and
- weakening of base and subgrade due to saturation from frequent ponding of long duration.

13.2 POLICY AND GUIDELINES

13.2.1 Introduction

Highway storm drainage facilities collect stormwater runoff and convey it through the roadway right-of-way in a manner that adequately drains the roadway and minimizes the potential for flooding and erosion to properties adjacent to the right-of-way. Storm drainage facilities consist of curbs, gutters, storm drains, channels and culverts. The placement and hydraulic capacities of storm drainage facilities should be designed to consider the potential for damage to adjacent property and to secure as low a degree of risk of traffic interruption by flooding as is consistent with the importance of the road, the design traffic service requirements and available funds.

Following is a summary of policies that should be followed for storm drain design and analysis. For a general discussion of policies and guidelines for storm drainage, the designer is referred to Reference *(1)*. For more design and engineering guidance, refer to AASHTO, *Highway Drainage Guidelines*, Chapter 9 *(2)*, and HEC 21 *(4)* and HEC 22 *(9)*.

13.2.2 Bridge Decks

Zero gradients, sag vertical curves and superelevation transitions with flat pavement sections should be avoided where possible on bridges. The minimum desirable longitudinal grade for bridge deck drainage is 0.5%. Many short, single-span bridges do not require drainage facilities. Where drainage facilities are needed, the preferred design is to place no inlets on the bridge deck and capture the runoff with inlets at the end(s) of the bridge. Quantity and quality of runoff should be maintained as required by applicable stormwater regulations. Section 13.7.7 discusses the maximum length of deck permitted without drainage facilities.

13.2.3 Curbs, Inlets and Flumes

Curbs or dikes, inlets and chutes or flumes are used where runoff from the pavement would erode fill slopes and/or to reduce right-of-way needed for shoulders or channels. Where storm drains are necessary, pavement sections are usually curbed.

13.2.4 Design Frequency

The design flood frequency for roadway drainage is related to the allowable water spread on the pavement and design speed. This design criteria is included in Section 13.9.

13.2.5 Detention Storage

Reduction of peak flows can be achieved by the storage of runoff in detention basins, storm drainage pipes, swales and channels and other detention storage facilities. Reduction of peak flows should be considered at locations where existing downstream conveyance facilities are inadequate to handle peak-flow rates from highway storm drainage facilities. In many locations, highway agencies or developers are not permitted to increase runoff over existing conditions, thus necessitating detention storage facilities. Additional benefits may include the reduction of downstream pipe sizes and the improvement of water quality by removing sediment and/or pollutants. See Chapter 12 — Storage Facilities.

13.2.6 Gutter Flow Calculations

Gutter flow calculations are necessary to relate the quantity of flow to the spread of water on the shoulder, parking lane or pavement section. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections. Refer to Section 13.10 for additional information and procedures.

13.2.7 Hydrology

The Rational method is the most common method in use for the design of storm drains when the momentary peak-flow rate is desired. Its use should be limited to systems with drainage areas of 300 acres or less. A minimum time of concentration of 7 min is generally acceptable. Drainage systems involving detention storage, pumping stations and large or complex storm systems require the development of a runoff hydrograph. The Rational method is briefly described in Section 13.6, and both the Rational method and hydrograph methods are described in Chapter 7, Hydrology.

13.2.8 Hydroplaning

See Section 13.8 for design criteria related to hydroplaning.

13.2.9 Inlets

The term "inlets" refers to all types of inlets (e.g., grate inlets, curb inlets, slotted inlets). Drainage inlets are sized and located to limit the spread of water on traffic lanes to tolerable widths for the design storm in accordance with the design criteria specified in Section 13.9. The width of water spread on the pavement at sags should not be substantially greater than the width of spread encountered on continuous grades.

Grate inlets and depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. All grate inlets shall be bicycle safe where used on roadways that allow bicycle travel. Curb inlets are preferred to grate inlets at major sag locations because of their debris handling capabilities. When grate inlets are used at sag locations, assume that they are half plugged with debris and size accordingly.

In locations where significant ponding may occur (e.g., at underpasses or sag vertical curves in depressed sections), recommended practice is to place flanking inlets on each side of the inlet at the low point in the sag (see Section 13.12.8). Review Section 13.11.3 for a discussion on the location of inlets.

13.2.10 Access Holes

The maximum spacing of access structures, whether access holes, junction boxes or inlets, should be as specified in Section 13.13.2. Refer to Section 13.13 for additional access hole criteria.

13.2.11 Pavement Drainage

Desirable longitudinal gutter grades should not be less than 0.5% for curbed pavements with a minimum of 0.3%. Cross slope considerations are discussed in Section 13.7.3.

13.2.12 Roadside and Median Ditches

Large amounts of runoff should be intercepted before it reaches the highway to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water that must be carried in the gutter section. Slope median areas and inside shoulders to a center depression to prevent runoff from the median area from running across the pavement. Surface channels should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard. Where permitted by the design velocities, channels should have a vegetative lining. Appropriate linings may be necessary where vegetation will not control erosion. Right-of-way restrictions/costs in urban areas often render impracticable the provision of roadside ditches.

13.2.13 <u>Storm Drains</u>

A storm drain is defined as that portion of the storm drainage system that receives runoff from inlets and conveys the runoff to some point where it is then discharged into a channel, water body or piped system. It consists of one or more pipes connecting two or more inlets. A storm drain may be a closed-conduit, open-conduit or some combination of the two. They may be designed with future development in mind, if appropriate. The storm drain system for a major sag vertical curve should use a higher design frequency (or return interval) to decrease the depth of ponding on the roadway and bridges. Where feasible, the storm drains shall be designed to avoid existing utilities. Attention shall be given to the storm drain outfalls to ensure that the potential for erosion is minimized. Drainage system design should be coordinated with the proposed staging of large construction projects to maintain an outlet throughout the construction project.

The placement and capacities should be consistent with local stormwater management plans. To assure self cleaning action, storm drains of 48-inch or less diameter shall demonstrate a calculated minimum design velocity of 2.5 ft/sec when flowing full. Exception will only be allowed upon written approval of the Region Director."

System planning prior to commencing the design of a storm drain system is essential. The basics required are discussed in Section 13.5 and include the general design approach, type of data required, information on initiating a cooperative agreement with a municipality, the importance of a preliminary sketch and some special considerations.

13.3 SYMBOLS AND DEFINITIONS

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Table 13-1 will be used. These symbols were selected because of their wide use in storm drainage publications.

13.4 CONCEPT DEFINITIONS

Following are discussions of concepts that will be important in a storm drainage analysis and design. These concepts will be used throughout the remainder of this Chapter in addressing different aspects of storm drainage analysis:

Check Storm — The use of a less frequent event (e.g., a 50-yr storm) to assess hazards at critical locations where water can pond to appreciable depths is commonly referred to as a check storm or check event.

Combination Inlet — A drainage inlet usually composed of a curb-opening inlet and a grate inlet.

Crown — The crown, sometimes known as the soffit, is the top inside of a pipe.

TABLE 13-1 — Symbols and Definitions

Symbol	Definition	Units
A	Area of cross section	ft^2
A	Watershed area	ac
a	Depth of depression	in
C	Runoff coefficient or coefficient	-
d	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
E_o	Ratio of frontal flow to total gutter flow (Q_w/Q)	-
h	Height of curb-opening inlet	ft
H	Head loss	ft
I	Rainfall intensity	in/h
K	Coefficient	-
L	Length of curb-opening inlet	ft
L	Pipe length	ft
L	Pavement width	ft
L	Length of runoff travel	ft
n	Roughness coefficient in Manning's formula	-
P	Perimeter of grate opening, neglecting bars and side against curb	ft
P	Tire pressure	lb/in^2
Q	Rate of discharge in gutter	ft^3/s
Q_i	Intercepted flow	ft^3/s
Q_s	Gutter capacity above the depressed section (see Figure 13-1)	ft^3/s
Q_T	Total flow	ft^3/s
Q_W	Gutter capacity in the depressed section (see Figure 13-1)	
R_h	Hydraulic radius	ft
S or S_x	Pavement cross slope	ft/ft
S	Crown slope of pavement	ft/ft
S or S_L	Longitudinal slope of pavement	ft/ft
S_w	Depressed section slope (see Figure 13-1)	ft/ft
T	Top width of water surface (spread on pavement)	ft
t_c	Time of concentration	min
T_D	Tire tread depth	in
T_s	Spread above depressed section	ft
TXD	Pavement texture depth	in
V	Vehicle speed	mi/h
V	Velocity of flow	ft/s
W	Width of depression for curb-opening inlets	ft
W_d	Rotational velocity on dry surface	rpm
WD	Water depth	in
W_w	Rotational velocity on flooded surface	rpm
y	Depth of flow in approach gutter	ft
Z	T/d, reciprocal of the cross slope	-

Culvert — A culvert is a closed conduit whose purpose is to convey surface water under a roadway, railroad or other impediment. It may have one or two inlets connected to it to convey drainage from the median area.

Curb-Opening — A drainage inlet consisting of an opening in the roadway curb.

Drop Inlet — A drainage inlet with a horizontal or nearly horizontal opening.

Equivalent Cross Slope — An imaginary straight cross slope having conveyance capacity equal to that of the given compound cross slope.

Flanking Inlets — Inlets placed upstream and on either side of an inlet at the low point in a sag vertical curve. These inlets intercept debris as the slope decreases and act in relief of the inlet at the low point.

Flow — Flow refers to a quantity of water that is flowing.

Frontal Flow — The portion of the flow that passes over the upstream side of a grate.

Grate Inlet — A drainage inlet composed of a grate in the roadway section or at the roadside in a low point, swale or channel.

Grate Perimeter — The sum of the lengths of all sides of a grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir-flow computations.

Gutter — That portion of the roadway section adjacent to the curb that is utilized to convey stormwater runoff. A composite gutter section consists of the section immediately adjacent to the curb, usually 2 ft at a cross-slope of, say 0.06 ft/ft, and the parking lane, shoulder or pavement at a cross slope of a lesser amount, say 0.02 ft/ft. A uniform gutter section has one constant cross slope. See Section 13.10 for additional information.

Hydraulic Grade Line — The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).

Inlet Efficiency — The ratio of flow intercepted by an inlet to total flow in the gutter.

Invert — The invert is the inside bottom of the pipe.

Lateral Line — A lateral line, sometimes referred to as a lead, has inlets connected to it but has no other storm drains connected. It is usually 24 in or less in diameter and is tributary to the trunk line.

Pressure Head — Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Runby/Bypass — Carryover flow that bypasses an inlet on grade and is carried in the street or channel to the next inlet downgrade. Inlets can be designed to allow a certain amount of runby for one design storm and larger or smaller amounts for other storms.

Sag Point/Major Sag Point — A low point in a vertical curve. A major sag point refers to a low point that can overflow only if water can pond to a depth of 1.5 ft or more.

Scupper — A vertical hole through a bridge deck for deck drainage. Sometimes, a horizontal opening in the curb or barrier is called a scupper.

Side-Flow Interception — Flow that is intercepted along the side of a grate inlet, as opposed to frontal interception.

Slotted Drain Inlet — A drainage inlet composed of a continuous slot built into the top of a pipe that serves to intercept, collect and transport the flow. Two types in general use are the vertical riser and the vane type.

Storm Drain — A storm drain is a closed or open conduit that conveys stormwater that has been collected by inlets to an adequate outfall. It generally consists of laterals or leads and trunk lines or mains. Culverts connected to the storm drainage system are considered part of the system.

Splash-Over — Portion of frontal flow at a grate that skips or splashes over the grate and is not intercepted.

Spread — The width of stormwater flow in the gutter measured laterally from the roadway curb. See Section 13.10 for methodology to compute.

Trunk Line — A trunk line is the main storm drain line. Lateral lines may be connected at inlet structures or access holes. A trunk line is sometimes referred to as a “main.”

Velocity Head — Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water ($V^2/2g$).

13.5 SYSTEM PLANNING

13.5.1 Introduction

The design of any storm drainage system involves the accumulation of basic data, familiarity with the project site and a basic understanding of the hydrologic and hydraulic principles and drainage policies associated with that design.

13.5.2 General Design Approach

The design of a storm drain system is generally a process that evolves as a project develops. The primary ingredients to this process are listed below in a general sequence by which they may be implemented. This *Manual* does not attempt to name all the participants of this process because it will vary with each Agency; however, the Hydraulics Engineer will play a major role:

1. Collect data (see Chapter 6 — Data Collection and Section 13.5.3).
2. Coordinate with other agencies (Section 13.5.4).
3. Prepare preliminary sketch (Section 13.5.5).
4. Determine inlet location and spacing (Sections 13.11 & 13.12).
5. Plan layout of storm drain system:

- locate main outfall,
- determine direction of flow,
- locate existing utilities,
- locate connecting mains, and
- locate access holes.

6. Size the pipes (Section 13.14).
7. Review hydraulic grade line (Section 13.15).
8. Prepare the plan.
9. Provide documentation (Chapter 4).

13.5.3 Required Data

The designer should be familiar with land-use patterns, the nature of the physical development of the area(s) to be served by the storm drainage system, the stormwater management plans for the area and the ultimate pattern of drainage (both overland and by storm drains) to some existing outfall location. Furthermore, there should be an understanding of the nature of the outfall because it usually has a significant influence on the storm drainage system. In environmentally sensitive areas, there may be water quality requirements to consider as well.

Actual surveys of these and other features are the most reliable means of gathering the required data. Photogrammetric mapping has become one of the most important methods of obtaining the large amounts of data required for drainage design, particularly for busy urban roadways with all the attendant urban development. Existing topographic maps, available from USGS, NRCS, many municipalities, some county governments and even private developers, are also valuable sources of the data needed for a proper storm drainage design. Developers and governmental planning agencies should be consulted regarding plans for the area in question. Often, in rapidly growing urban areas, the physical characteristics of an area to be served by a storm drainage system may change drastically in a very short time. In such cases, the designer is to anticipate these changes and consider them in the storm drainage design. Comprehensive Stormwater Management Plans and Floodplain Ordinances should be reviewed when they are available.

13.5.4 Cooperative Projects

Cooperative storm drain projects with cities and municipalities may be beneficial where both a mutual economic benefit and a demonstrated need exists. Early coordination with the governmental entity involved is necessary to determine the scope of the project. Each cooperative project may be initiated by a resolution adopted by the governing body of the municipality either (1) requesting the improvements and/or indicating its willingness to share the cost of a State project, or (2) indicating the municipality’s intention to make certain improvements and requesting State cost participation in the municipal project.

13.5.5 Preliminary Sketch

Preliminary sketches or schematics, featuring the basic components of the intended design, are useful to the designer. Such sketches should indicate watershed areas and land use, existing drainage patterns, plan and profile of the roadway, street and driveway layout with respect to the project roadway, underground utility locations and elevations, locations of proposed retaining walls, bridge abutments and piers, logical inlet and access hole locations, preliminary lateral and trunk line layouts and a clear definition of the outfall location and characteristics. This sketch should be reviewed with the traffic staging plans and soils recommendations for areas that are incompatible with required construction staging. With this sketch or schematic, the designer is able to proceed with the detailed process of storm drainage design calculations, adjustments and refinements.

Unless the proposed system is very simple and small, the designer should not ignore a preliminary plan as described above. Upon completion of the design, documentation of the overall plan is facilitated by the preliminary schematic.

13.5.6 Location and Size of Storm Drain

Storm-drain pipes should not decrease in size in a downstream direction regardless of the available pipe gradient.

Locate the storm drain to avoid conflicts with utilities, foundations or other obstacles. Minimizing the depth of the storm drain may produce a significant cost savings. Coordination with utility owners during the design phase is necessary to determine if an adjustment to the utilities or the storm drain system is required. The location of the storm drain may affect construction activities and phasing. The storm drain should be located to minimize traffic disruption during construction. Dual trunklines along each side of the roadway may be used in some cases where it is difficult or more costly to install laterals. Temporary drainage may be needed to avoid increases in flood hazards during construction.

13.5.7 Outfall Policy

Urban highway systems may increase peak discharge and volume due to increases in the impervious area and decreases in the time of concentration or lag time. Accumulation or diversion of flow may also result in an increase in runoff at storm drain outfalls. The following shall apply for all outfall in urban areas:

- A significant increase in peak discharge runoff is defined as a 5% increase in peak discharge above conditions existing at the time of design for the 2-, 5-, 10-, 25-, 50- and 100-yr storm or flood frequencies.

- For new highway projects, make provisions to limit significant increases in the peak discharge and volume of storm drain discharge to avoid increases in downstream flood hazards and environmental impacts.

- Make hydrologic computations to estimate the increase in runoff due to new highway construction. The computations will include the 2-, 5-, 10-, 25-, 50- and 100-yr storm or flood frequencies.

- Require detention ponding or increased capacity of the outfall drainage system for significant increases in peak discharge above preconstruction conditions. The detention pond will be designed to limit peak outflows to predevelopment conditions for all flood frequencies as detailed in Chapter 12 of this *Manual*. If detention is not provided to limit a significant increase in peak discharge, the outfall will be increased in size to accommodate the increase in discharge.

- Require detailed floodplain mapping of the outfall channel if the drainage system increases the water surface elevation more than 0.2 ft. The detailed floodplain map of the outfall channel will be submitted to the governing planning agency and FEMA.

- Land developers must limit increases in runoff from new developments by installing detention ponds or by increasing the outfall or highway drainage system capacity. If an analysis indicates that the highway drainage system is undersized for existing levels of development, the Highway Agency may elect to participate in a project to bring the drainage system up to current standards.

Limiting detention peak outflow only for large storm or flood events could result in uncontrolled increases in runoff for the smaller, more frequent storms that may be undesirable:

- The developers will provide detention ponds or larger outfall systems to accommodate the increase in runoff due to the development. Detention pond outlets must be designed to limit runoff for a full range of storm or flood frequencies as detailed in Chapter 12 of this *Manual*.

- In accordance to Utah Law, city and county agencies must notify the Utah Department of Transportation for proposed developments that may increase runoff to the State highway system. City and county governments must require the developers to either limit the increases in runoff equal to or less than predevelopment conditions or to provide a larger outfall or highway drainage system that will accommodate increases in runoff. The developers will assume all liability for damages that may be attributed to uncontrolled increases in runoff due to development.

A change in the timing of peak discharges from highway or other developments may result in an increase in the peak discharge of the receiving waters. A detention pond may limit runoff peaks from a development to existing conditions at the outfall, but changes in the peak timing may result in an increase in peak discharge and stage elevation of the receiving waters. A hydrologic analysis of the receiving waters will provide useful information in assessing flood hazard impacts due to changes in peak timing. Hydrograph methods are the most reliable when estimating the effects of changes in peak runoff rates in receiving waters due to detention ponds located on tributaries:

- For major storm drain systems, the hydrologic analysis must include the effects of any uncontrolled increases in runoff to the downstream drainage systems. A hydrologic analysis of the receiving waters is required to determine if changes in peak discharge timing from detention ponds will increase the peak discharge in the receiving waters.

- Table 13-7 (in Section 13.15) on coincidental probabilities may be used to determine the storm frequency to be used to estimate the effects of changes in peak timing if the drainage area of the tributary controlled by the detention pond is significantly less than that of the receiving waters. If the analysis indicates a significant increase in the peak discharge of the receiving waters, the detention must be designed to decrease the peak outflow discharge until there is no significant increase in the peak discharge of the receiving waters.

- For detention ponds exceeding 0.3 ac•ft, a hydrograph method other than the Rational method is required to model the detention pond and the receiving waters.

The channel stability of outfall channels must be assessed when there are significant changes in discharges due to highway projects or developments. A detention pond may limit the peak discharge to predevelopment conditions for the larger flood frequencies; however, a significant increase in the lower frequencies (e.g., the 2-yr discharge) could significantly impact channel stability. An increase in the 1.1- to 2.5-yr frequency discharges may increase the sediment transport rate such that changes in channel stability may occur. Chapter 8 provides guidance on channel stability. Chapter 17 provides guidelines for channel protection measures.

Outfall channel stability assessment must be made for significant increases in peak discharge. Channel stability measures must be employed if found to be necessary.

13.6 HYDROLOGY

13.6.1 Introduction

The Rational method is the most common method in use for the design of storm drains when the momentary peak-flow rate is desired. Its use should be limited to systems with drainage areas of 300 ac or less. Drainage systems involving detention storage and pumping stations require the development of a runoff hydrograph. Both methods are described in Chapter 7, Hydrology.

13.6.2 Rational Method

The Rational Equation is written as follows:

$$Q = CIA = \left(\sum CA\right) I \tag{13.1}$$

where:
- Q = discharge, ft^3/s
- C = runoff coefficient (see Section 7.18)
- I = rainfall intensity, in/h (see IDF curves in Chapter 7)
- A = drainage area, ac

13.6.2.1 Runoff Coefficient

The runoff coefficients for various types of surfaces are discussed in Chapter 7, Section 7.18, with tables of appropriate values. The weighted C value is to be based on a ratio of the drainage areas associated with each C value as follows:

$$\text{Weighted } C = [A_1C_1 + A_2C_2 + A_3C_3] / [A_1 + A_2 + A_3]$$

13.6.2.2 Rainfall Intensity

Rainfall intensity is the intensity of rainfall in inches per hour for a duration equal to the time of concentration. Intensity is a rate of rainfall over an interval of time such that intensity multiplied by duration equals amount of rain; i.e., an intensity of 5 in/h for a duration of 5 min indicates a total rainfall amount of (5)(5/60) = 0.4 in. See Chapter 7, Hydrology for a more complete discussion and data to be used for determining the intensity of rainfall.

13.6.2.3 Time of Concentration

The time of concentration is defined as the period required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. The designer is usually concerned with two different times of concentration — one for inlet spacing and the other for pipe sizing. There is a major difference between the two times.

13.6.2.3.1 Inlet Spacing

The time of concentration (t_c) for inlet spacing is the time for water to flow from the hydraulically Gmost distant point of the drainage area to the inlet, which is known as the inlet time. Usually this is the sum of the time required for water to move across the pavement or overland back of the curb to the gutter, plus the time required for flow to move through the length of gutter to the inlet. For pavement drainage, when the total time of concentration to the upstream inlet is less than 7 min, a minimum t_c of 7 min should be used to estimate the intensity of rainfall. The time of concentration for the second downstream inlet and each succeeding inlet should be determined independently, the same as the first inlet. For a constant roadway grade and relatively uniform contributing drainage area, the time of concentration for each succeeding inlet could also be constant.

13.6.2.3.2 Pipe Sizing

The time of concentration for pipe sizing is defined as the time required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. In storm drain applications, time of concentration generally consists of two components: (1) the time to flow to the inlet, which can consist of sheet flow, shallow concentrated flow and channel or gutter flow segments, and (2) the time to flow through the storm drain to the point under consideration. However, some NRCS Curve Number methods combine segments into a single lag time that is then empirically related to an overland time of concentration.

The sheet flow time of concentration segment is typically developed using the kinematic wave approach. The NRCS Velocity Method provides a means to compute sheet flow and shallow concentrated flow travel time segments. Channel and storm drain times of concentration can be developed using Manning's equation or the HEC 22 *(9)* triangular gutter approach. Travel time within each component and storm drain pipes can be estimated by the relation:

$$t_t = L / 60V \tag{13.2}$$

where:
- t_t = travel time, min
- L = length of pipe in which runoff must travel, ft
- V = estimated or calculated normal velocity, ft/s

All of these methods are further described in Chapter 7, Hydrology. To summarize, the time of concentration for any point on a storm drain is the inlet time for the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in the storm drainage system, the longest t_c is used to estimate the intensity (I). There could be exceptions to this generality, for example, where there is a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c. The designer should be cognizant of this possibility when joining drainage areas and determine which drainage area governs. To determine which drainage area controls, compute the peak discharge for each t_c. Note that, when computing the peak discharge with the shorter t_c, not all the area from the basin with the longest t_c will contribute runoff. One way to estimate the contributing area, A_c, is as follows:

$$A_c = A\,[t_{c1} / t_{c2}] \tag{13.3}$$

Where $t_{c1} < t_{c2}$, and A is the area of the basin with the longest t_c.

In municipal areas, a minimum time of concentration of 5 min is recommended for calculation of runoff from paved areas; all other areas should be calculated on a case-by-case basis.

13.6.3 Other Hydrologic Methods

The use of other hydrologic methods may be desirable for the following reasons:

- provide compatibility with studies of systems adjacent to or connected with the storm drain system;
- as required by government agencies;
- produce a more detailed analysis;
- provide a more realistic simulation of detention ponding, channel routing or pipe routing; and
- provide an analysis of peak timing conditions of receiving waters.

Methods such as NRCS, USGS Urban hydrology procedure, Colorado Urban Hydrograph Procedure, Santa Barbara Urban Hydrograph Procedure or kinematic wave method are commonly used for more complex conditions.

13.6.4 Detention

Estimation of the effects of detention requires a reservoir routing procedure such as that presented in Chapter 12 – Storage Facilities. By introducing detention ponds, the designer is able to attenuate the peak of the runoff hydrograph, thus reducing the outflow design discharge rate. The same approach for peak-flow attenuation is valid and particularly useful in a storm drainage system in which there are substantial lengths of large diameter pipes. In such systems, the storage capacity of the pipes can have a substantial effect on the final shape of the runoff hydrograph.

13.7 PAVEMENT DRAINAGE

13.7.1 Introduction

Roadway features considered during gutter, inlet and pavement drainage calculations include:

- longitudinal and cross slope,
- curb and gutter sections,
- pavement texture/surface roughness,
- roadside and median ditches, and
- bridge decks.

The pavement width, cross slope, profile and pavement texture control the time it takes for stormwater to drain to the gutter section. The gutter cross section and longitudinal slope control the quantity of flow that can be carried in the gutter section.

13.7.2 Longitudinal Slope

A minimum longitudinal gradient is more important for a curbed pavement than for an uncurbed pavement because it is susceptible to the spread of stormwater against the curb. Flat gradients on uncurbed pavements can also lead to a spread problem if vegetation is allowed to build up along the pavement edge.

Desirable gutter grades should be greater than 0.3% for curbed pavements. Minimum grades can be maintained in very flat terrain by use of a rolling profile.

To provide adequate drainage in sag vertical curves, a minimum slope of 0.3% should be maintained within 50 ft of the level point in the curve. This is accomplished where the length of the curve divided by the algebraic difference in grades is equal to or less than 167 (ft/%). Although ponding is not usually a problem at crest vertical curves, on extremely flat curves a similar minimum gradient should be provided to facilitate drainage.

13.7.3 Cross Slope

Reference *(1)* is standard practice and should be consulted prior to deviation from this *Manual*.

The design of pavement cross slope is often a compromise between the need for reasonably steep cross slopes for drainage and relatively flat cross slopes for driver comfort. Reference *(8)* reports that cross slopes of 2% have little effect on driver effort in steering, especially with power steering or on friction demand for vehicular stability. Use of a cross slope steeper than 2% on pavements with a central crown line is not desirable. In areas of intense rainfall, a somewhat steeper cross slope may be necessary to facilitate drainage. In such areas, the cross slope may be increased to 2.5%.

When three or more lanes are inclined in the same direction on multilane pavements, it is desirable that each successive pair of lanes, or the portion thereof outward from the first two lanes from the crown line, have an increased slope. The two lanes adjacent to the crown line should be pitched at the normal slope and successive lane pairs, or portions thereof outward, should be increased by approximately 0.5% to 1%. Where three or more lanes are provided in each direction, the maximum pavement cross slope should be limited to 4%.

It is desirable to provide a break in cross slope at two lanes, with three lanes the upper limit. Although not widely encouraged, inside lanes can be sloped toward the median. This should not be used unless four continuous lanes or some physical constraint on the roadway elevations occurs, because inside lanes are used for high-speed traffic and the allowable water depth is lower. Median areas should not be drained across traveled lanes. A careful check should be made of designs to minimize the number and length of flat pavement sections in cross slope transition areas, and consideration should be given to increasing cross slopes in sag vertical curves and crest vertical curves and in sections of flat longitudinal grades. Where curbs are used, depressed gutter sections can be effective at increasing gutter capacity and reducing spread on the pavement.

13.7.4 Pavement Texture

The pavement texture is an important consideration for roadway surface drainage. Although the hydraulic design engineer will have little control over the selection of the pavement type or its texture, it is important to know that pavement texture does have an impact on the buildup of water depth on the pavement during rain storms. A good macrotexture provides a channel for water to escape from the tire/pavement interface and, thus, reduces the potential for hydroplaning.

A high level of macrotexture may be achieved by tinning new portland cement concrete pavements while it is still in the plastic state. Re-texturing of an existing portland cement concrete surface can be accomplished through pavement grooving and cold milling. Both longitudinal and transverse grooving are very effective in achieving macrotexture in concrete pavement. Transverse grooving aids in surface runoff resulting in less wet pavement time. Combinations of longitudinal and transverse grooving provide the most adequate drainage for high-speed conditions.

13.7.5 Curb and Gutter

Curbing at the outside edge of pavements is normal practice for low-speed, urban highway facilities. They serve several purposes that include containing the surface runoff within the roadway and away from adjacent properties, preventing erosion, providing pavement delineation and enabling the orderly development of property adjacent to the roadway. Curbs may be either barrier or mountable type, and they are typically portland cement concrete, although bituminous curb is used occasionally. Barrier curbs range in height from 6 in to 10 in with a batter of 1 in per 3 in of height. Mountable curbs are less than 6 in in height and have rounded or plane-sloping faces. Gutters are available in 1 ft through 3 ft widths. A curb and gutter forms a triangular channel that can be an efficient hydraulic conveyance facility that can convey runoff of a lesser magnitude than the design flow without interruption of the traffic. When a design storm flow occurs, there is a spread or widening of the conveyed water surface and the water spreads to include, not only the gutter width, but also parking lanes or shoulders and portions of the traveled surface. This is the width the hydraulics engineer is most concerned with in curb and gutter flow, and limiting this width becomes a very important design criterion. This will be discussed in further detail in Section 13.9.

Where practicable, it is desirable to intercept runoff from cut slopes and other areas draining toward the roadway before it reaches the highway, to minimize the deposition of sediment and other debris on the roadway and to reduce the amount of water that must be carried in the gutter section. Shallow swale sections at the edge of the roadway pavement or shoulder offer advantages over curbed sections where curbs are not needed for traffic control. These advantages include a lesser hazard to traffic than a near-vertical curb and hydraulic capacity that is not dependent on spread on the pavement. These swale sections without curbs are particularly appropriate where curbs have historically been used to prevent water from eroding fill slopes.

13.7.6 Roadside and Median Channels

Roadside channels are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas that drain toward the highway. Due to right-of-way limitations, roadside channels cannot be used on most urban arterials. They can be used in cut sections, depressed sections and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted in the ditch as appropriate.

It is preferable to slope median areas and inside shoulders to a center swale to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.

13.7.7 Bridge Decks

Drainage of bridge decks is similar to other curbed roadway sections. It is often less efficient, because cross slopes are flatter, parapets collect large amounts of debris, and small drainage inlets or scuppers have a higher potential for clogging by debris. Bridge deck construction usually requires a constant cross slope. Because of the difficulties in providing and maintaining adequate deck drainage systems, gutter flow from roadways should be intercepted before it reaches a bridge. In many cases, deck drainage must be carried several spans to the bridge end for disposal.

The gutter spread should be checked to ensure compliance with the design criteria in Section 13.9. Zero gradients and sag vertical curves should be avoided on bridges. The minimum desirable longitudinal slope for bridge deck drainage should be 0.5%. In many areas of the nation, scuppers are the recommended method of deck drainage because they can reduce the problems of transporting a relatively large concentration of runoff in an area of generally limited right-of-way. They also have a low initial cost and are relatively easy to maintain. However, the use of scuppers should be evaluated for site-specific concerns. Scuppers should not be located over embankments, slope pavement, slope protection, navigational channels, driving lanes or railroad tracks. Runoff collected and transported to the end of the bridge should generally be collected by inlets and down drains, although sod flumes may be used for extremely minor flows in some areas. Runoff should also be handled in compliance with applicable stormwater quality regulations.

Many bridges will not require any drainage structures at all. To determine the length of deck permitted without drainage structures and without exceeding the allowable spread, the following equation, which is based on a uniform cross slope, can be utilized *(8)*:

$$L = \frac{137\,400(S_x^{1.67})(S^{0.5})(T^{2.67})}{CnIW} \qquad (13.4)$$

where:
- L = length of deck, ft
- S = longitudinal slope, ft/ft
- S_x = cross slope, ft/ft
- W = width of drained deck, ft
- C = runoff coefficient
- I = rainfall intensity, in/h
- n = Manning's n
- T = allowable spread, ft

13.7.8 Shoulder Gutter and/or Curbs

Shoulder gutter and/or curbs may be appropriate to protect fill slopes from erosion caused by water from the roadway pavement. They are recommended on fill slopes higher than 20 ft – standard slopes are 1V:2H. They are also recommended on fill slopes higher than 10 ft – standard slopes are 1V:6H and 1V:3H, if the roadway grade is greater than 2%. In areas where permanent vegetation cannot be established, shoulder gutter and/or curbs are recommended on fill slopes higher than 10 ft regardless of the grade. Inspection of the existing/proposed site conditions and contact with maintenance and construction personnel shall be made by the designer to determine if vegetation will survive.

Shoulder gutter and/or curbs may be appropriate at bridge ends where concentrated flow from the bridge deck would otherwise run down the fill slope. This section of gutter should be long enough to include the transitions. Shoulder gutters are not required on the high side of super-elevated sections or adjacent to barrier walls on high fills.

13.7.9 Median/Median Barriers

Medians are commonly used to separate opposing lanes of traffic on divided highways. It is preferable to slope median areas and inside shoulders to a center depression to prevent drainage from the median area from running across the traveled pavement. Where median barriers are used and, particularly on horizontal curves with associated superelevations, it is necessary to provide inlets and connecting storm drains to collect the water that accumulates against the barrier. Slotted drains adjacent to the median barrier and, in some cases, weep holes in the barrier can also be used for this purpose.

13.7.10 Impact Attenuators

The location of impact attenuator systems should be reviewed to determine the need for drainage structures in these areas. With impact attenuator systems, it is necessary to have a clear or unobstructed opening as traffic approaches the point of impact to allow a vehicle to impact the system head on. If the impact attenuator is placed in an area where superelevation or other grade separation occurs, grate inlets and/or slotted drains may need to be placed to prevent water from running through the clear opening and crossing the highway lanes or ramp lanes. Curb, curb-type structures or swales cannot be used to direct water across this clear opening because vehicular vaulting could occur when the impact attenuator system is utilized.

13.8 HYDROPLANING

Reference *(10)* suggests that hydroplaning conditions can develop for relatively low vehicular speeds and at low rainfall intensities for storms that frequently occur each year. Analysis methods developed through this research effort provide guidance in identifying potential hydroplaning conditions. Unfortunately, it is virtually impossible to prevent water from exceeding a depth that would be identified through this analysis procedure as a potential hydroplaning condition for wide pavements during high-intensity rainfall and under some relationship of the primary controlling factors of:

- vehicular speed;
- tire conditions (pressure and tire tread);
- pavement micro and macrotexture;
- roadway geometrics (pavement width, cross slope, grade); and
- pavement conditions (rutting, depressions, roughness).

Speed appears as a significant factor in the occurrence of hydroplaning; therefore, it is considered to be the driver’s responsibility to exercise prudence and caution when driving during wet conditions *(2)*. In many respects, hydroplaning conditions are analogous to ice or snow on the roadway.

Designers do not have control over all factors involved in hydroplaning. However, many remedial measures can be included in development of a project to reduce hydroplaning potential. The following is provided as guidance for the designer as practical measures to consider in accordance with Reference *(11)*:

Pavement Sheet Flow

- maximize transverse slope,
- maximize pavement roughness, and
- use of graded course (porous pavements).

Gutter Flow

- limit street spread (by decreasing inlet spacing), and
- maximize interception of gutter flow above superelevation transitions.

Sag Areas

- limit pond duration and depth.

Overtopping

- limit depth and duration of overtopping flow.

If suitable measures cannot be implemented to address an area of high potential for hydroplaning, or an identified existing problem area, consideration should be given to installing advance warning signs.

13.9 DESIGN FREQUENCY AND SPREAD

13.9.1 Design Frequency

The design storm frequency for pavement drainage may not be consistent with the frequency selected for other components of the storm drain system. For example, a 10-yr return period may be selected to limit spread on grade and a 50-yr return period may be used at a sag location. The trunkline and laterals on grade may be sized for the 10-yr frequency where the trunkline or outfall from a sag area may be sized to accommodate a 50-yr return period.

13.9.2 Spread

In general, the spread should be held to the specified width for design frequencies. For storms of greater magnitude, the spread can be allowed to utilize “most” of the pavement as an open channel. For multi-laned curb and gutter, or guttered roadways with no parking, it is not practical to avoid travel lane flooding when longitudinal grades are flat (0.2% to 1%). However, flooding should not exceed the lane adjacent to the gutter (or shoulder) for design conditions. Municipal bridges with curb and gutter should also use this criterion. For single-lane roadways, at least 10 ft of roadway on each direction, should remain unflooded for design conditions.

13.9.3 Selection

The major considerations for selecting a design frequency and spread include highway classification, because it defines and reflects public expectations for finding water on the pavement surface. Ponding should be minimized on the traffic lanes of high-speed, high-volume highways where it is not expected.

Highway speed is another major consideration, because at speeds greater than 45 mph, even a shallow depth of water on the pavement can cause hydroplaning. Design speed is recommended for use in evaluating hydroplaning potential. It is clearly unreasonable and not cost effective to provide the same level of protection for low-speed facilities as for high-speed facilities.

Other considerations include inconvenience, hazards and nuisances to pedestrian traffic and buildings adjacent to roadways that are located within the splash zone. These considerations should not be minimized and, in some locations (e.g., commercial areas), may assume major importance.

13.9.4 Design Criteria

Roadway Classification		Design Frequency	Design Spread
High Volume	< 45 mph	10-yr	Shoulder + 3 ft
	> 45 mph	10-yr	Shoulder
	sag point	50-yr	Shoulder + 3 ft
Collector	< 45 mph	10-yr	½ driving lane
	> 45 mph	10-yr	Shoulder
	sag point	10-yr	½ driving lane
Local Streets	low ADT	5-yr	½ driving lane
	high ADT	10-yr	½ driving lane
	sag point	10-yr	½ driving lane

Note: These criteria apply to shoulder widths of 6 ft or greater. Where shoulder widths are less than 6 ft, a minimum design spread of 6 ft should be considered.

13.10 GUTTER FLOW CALCULATIONS

13.10.1 Introduction

Gutter flow calculations are necessary to relate the quantity of flow (Q) in the curbed channel to the spread of water on the shoulder, parking lane or pavement section. The nomograph on Figure 13-1 can be utilized to solve uniform cross slope channels, composite gutter sections and V-shape gutter sections. Figure 13-3 is also useful in solving composite gutter section problems. Computer programs, such as the HYDRAIN program *(6)*, is also useful for this computation and inlet capacity. Composite gutter sections have a greater hydraulic capacity for normal cross slopes than uniform gutter sections and are therefore preferred. Example problems for each gutter section are shown in the following sections.

13.10.2 Manning's n For Pavements

The roughness of the pavement surface affects water spread. The methods for determining spread provided in this Chapter use Manning's roughness coefficient (n). Refer to Table 13-2 for recommended values.

TABLE 13-2 — Manning's n for Street and Pavement Gutters

Type of Gutter or Pavement	Manning's n
Concrete Gutter, troweled finish	0.012
Asphalt Pavement: Smooth texture Rough texture	 0.013 0.016
Concrete Gutter, Asphalt Pavement Smooth Rough	 0.013 0.015
Concrete Pavement Float finish Broom finish	 0.014 0.016
For gutters with small slope, where sediment may accumulate, increase above n values by:	0.002

Source: Reference *(7).*

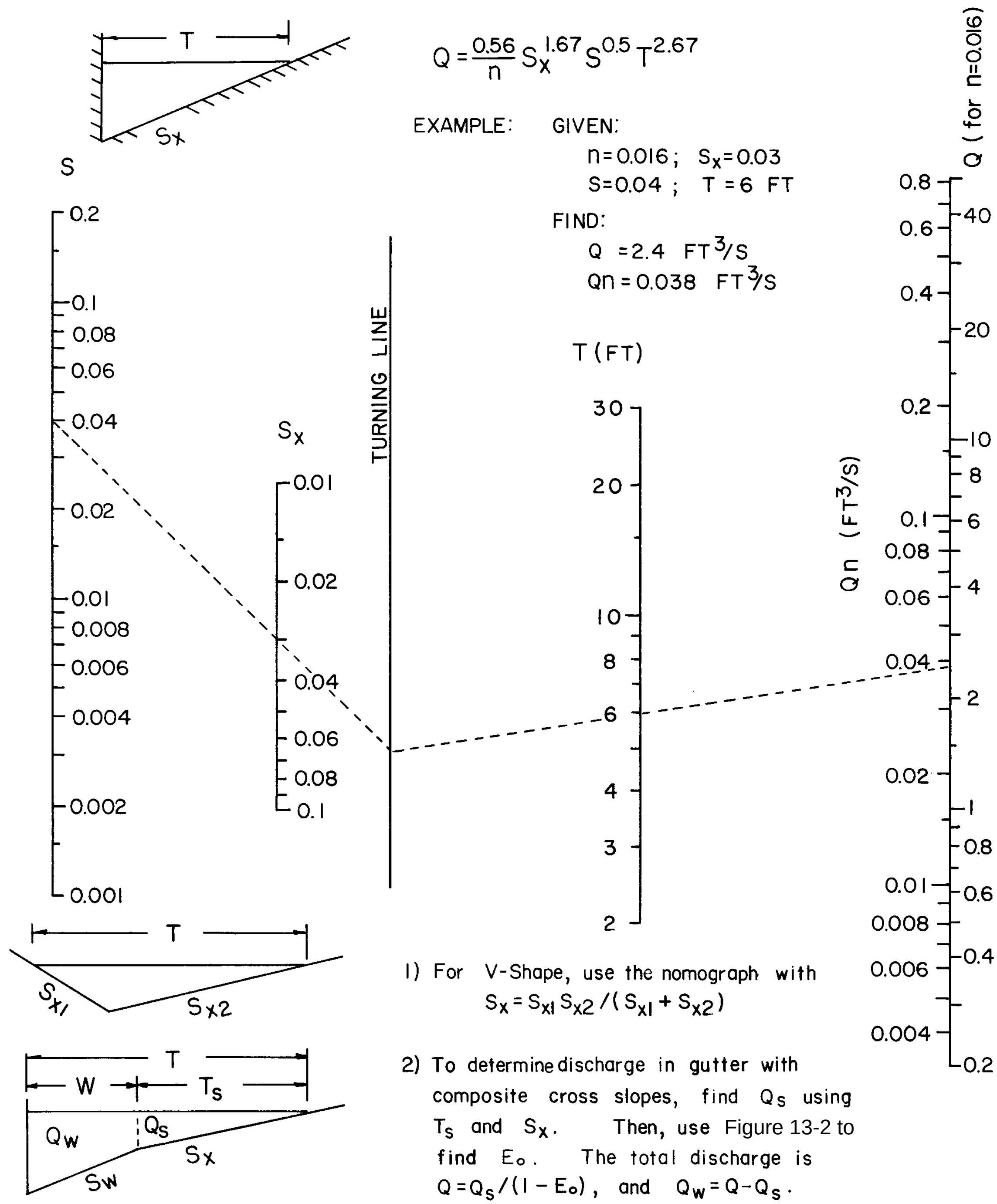
13.10.3 Uniform Cross Slope Procedure

The nomograph in Figure 13-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

CONDITION 1: Find spread, given gutter flow:

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), gutter flow (Q) and Manning's n.

Step 2 Draw a line between the S and S_x scales, and note where it intersects the turning line.



Flow in Triangular Gutter Sections - English Units

FIGURE 13-1 — Flow in Triangular Gutter Sections

Source: HEC 22 *(9)*.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

Step 4 Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

CONDITION 2: Find gutter flow, given spread:

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), spread (T) and Manning's n.

Step 2 Draw a line between the S and S_x scales, and note where it intersects the turning line.

Step 3 Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

Step 4 For Manning's n values of 0.016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n(Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

13.10.4 <u>Composite Gutter Sections Procedure</u>

Figure 13-2 can be used to find the flow in a gutter section with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets:

CONDITION 1: Find spread, given flow:

Step 1 Determine input parameters, including longitudinal slope (S), cross slope (S_x), depressed section slope (S_w), depressed section width (W), Manning's n, gutter flow (Q) and a trial value of the gutter capacity above the depressed section (Q_s). (Example: S = 0.01; S_x = 0.02; S_w = 0.06; W = 2 ft; n = 0.016; Q = 2.0 ft^3/s; try Q_s = 0.7 ft^3/s).

Step 2 Calculate the gutter flow in W (Q_w), using the equation:

$$Q_w = Q - Q_s \quad (Q_w = 2.0 - 0.7 = 1.3\ ft^3/s) \tag{13.5}$$

Step 3 Calculate the ratios Q_w/Q and S_w/S_x, and use Figure 13-2 to find an appropriate value of W/T. (Q_w/Q = 1.3/2.0 = 0.65; S_w/S_x = 0.06/0.02 = 3. From Figure 13-2, W/T = 0.27).

Step 4 Calculate the spread (T) by dividing the depressed section width (W) by the value of W/T from Step 3. (T = 2.0/0.27 = 7.41 ft).

Step 5 Find the spread above the depressed section (T_s) by subtracting W from the value of T obtained in Step 4. (T_s = 7.41 – 2.0 = 5.41 ft).

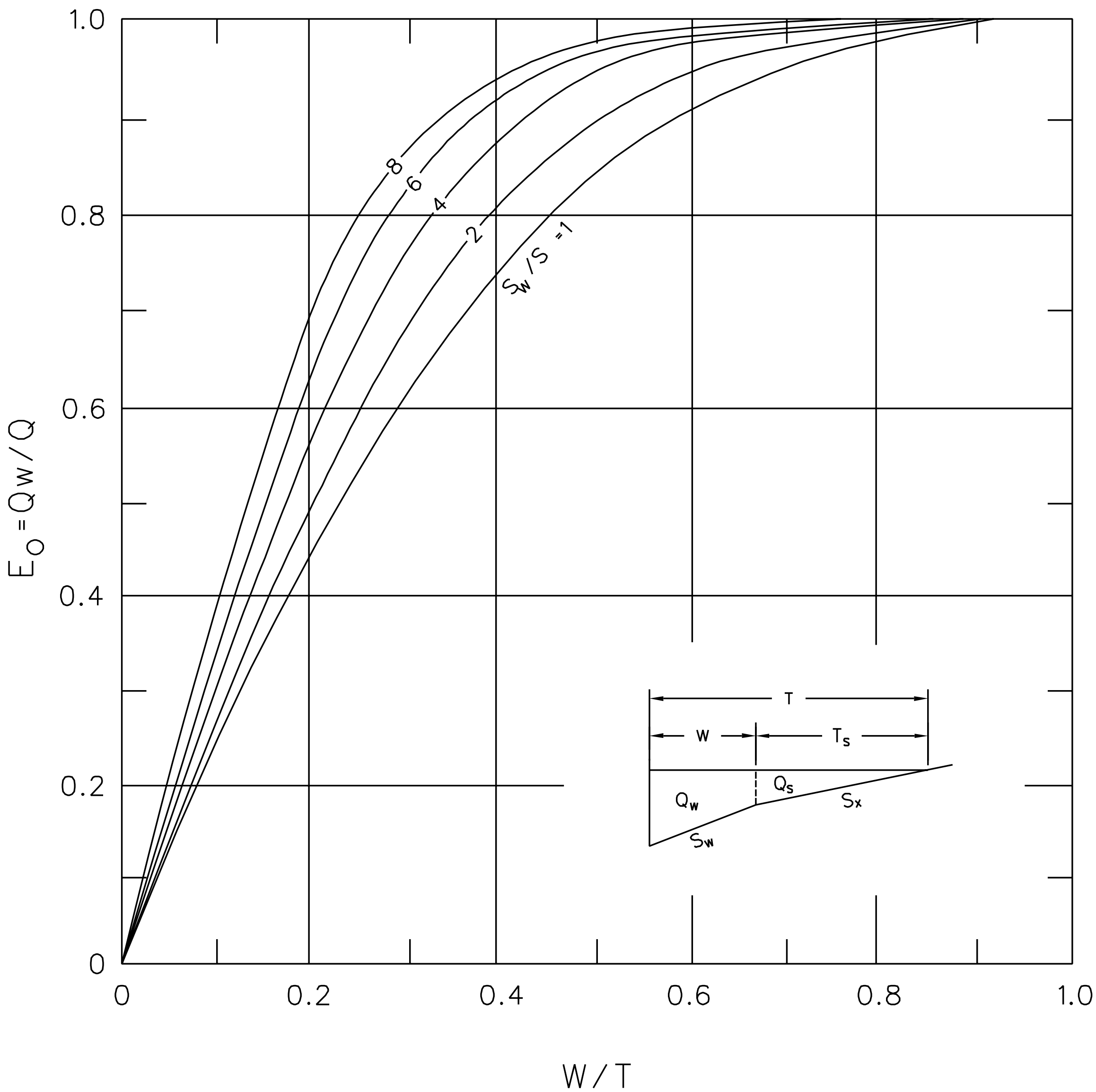


FIGURE 13-2 — Ratio of Frontal Flow to Total Gutter Flow

Source: HEC 22 *(9).*

Step 6 Use the value of T_s from Step 5 and Manning's n, S and S_x to find the actual value of Q_s from Figure 13-1. (From Figure 13-1, $Q_s = 0.5\ ft^3/s$).

Step 7 Compare the value of Q_s from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Q_s and return to Step 1:

(Compare 0.5 to 0.7 "no good." Try $Q_s = 0.8$; then 2.0 – 0.8 = 1.2, and 1.2/2.0 = 0.6. From Figure 13-2, W/T = 0.23; then T = 2.0/0.23 = 8.7 ft and T_s = 8.7 – 2.0 = 6.7 ft. From Figure 13-1, $Q_s = 0.8\ ft^3/s$ — OK).

ANSWER: Spread T = 8.7 ft

CONDITION 2: Find gutter flow, given spread:

Step 1 Determine input parameters, including spread (T), spread above the depressed section (T_s), cross slope (S_x), longitudinal slope (S), depressed section slope (S_w), depressed section width (W), Manning's n and depth of gutter flow (d):

EXAMPLE: Allowable spread, T = 10 ft; W = 2 ft; T_s = 10.0 – 2.0 = 8 ft; S_x = 0.04; S = 0.005 ft/ft; S_w = 0.06; n = 0.016; d = 0.43 ft.

Step 2 Use Figure 13-1 to determine the capacity of the gutter section above the depressed section (Q_s). Use the procedure for uniform cross slopes, Condition 2, substituting T_s for T. (From Figure 13-1, $Q_s = 3.0\ ft^3/s$).

Step 3 Calculate the ratios W/T and S_w/S_x and, from Figure 13-2, find the appropriate value of E_o (the ratio of Q_w/Q). (W/T = 2.0/10.0 = 0.2; S_w/S_x = 0.06/0.04 = 1.5; from Figure 13-1, E_o = 0.46).

Step 4 Calculate the total gutter flow using the equation:

$$Q = Q_s/(1 - E_o) \tag{13.6}$$

where: Q = gutter flow rate, ft^3/s

Q_s = flow capacity of the gutter section above the depressed section, ft^3/s

E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

($Q = 3.0 / (1 – 0.46) = 5.6\ ft^3/s$)

Step 5 Calculate the gutter flow width (W), using Equation 13.5:

($Q_w = Q – Q_s = 5.6 – 3.0 = 2.6\ ft^3/s$)

Note: Figure 13-3 can also be used to calculate the flow in a composite gutter section.

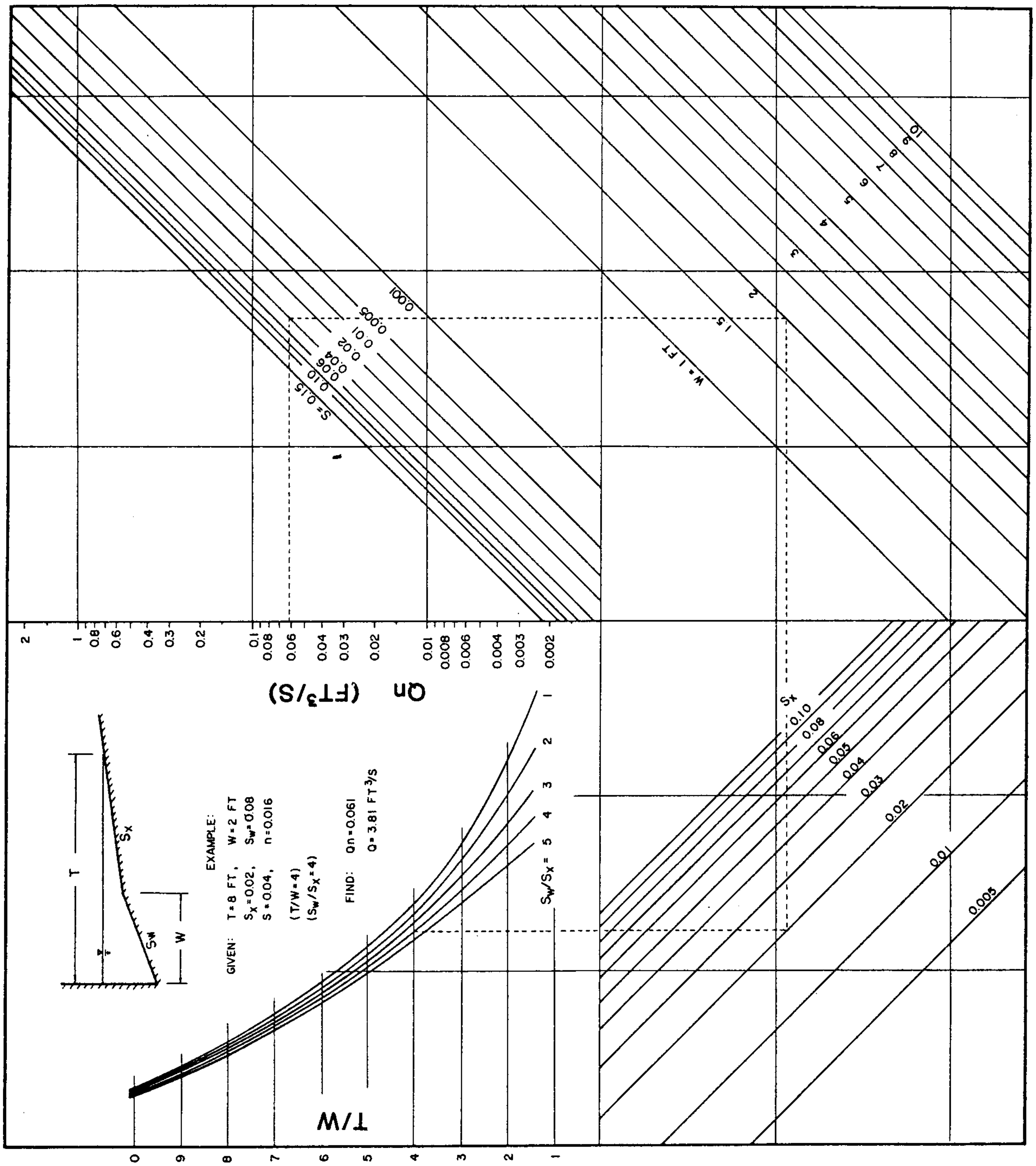


FIGURE 13-3 — Flow in Composite Gutter Sections

Source: HEC 12 (5).

13.10.5 V-Type Gutter Sections (Procedures)

Figure 13-1 can also be used to solve V-type channel problems. The spread (T) can be calculated for a given flow (Q), or the flow can be calculated for a given spread. This method can be used to calculate approximate flow conditions in the triangular channel adjacent to concrete median barriers. It assumes that the effective flow is confined to the V channel with spread T_1. Figure 13-4 illustrates the following procedure for a V-type gutter:

CONDITION 1: Given flow (Q), find spread (T):

Step 1 Determine input parameters, including longitudinal slope (S), cross slope $S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$, Manning's n, total flow (Q). (Example: S = 0.01, S_{x1} = 0.06, S_{x2} = 0.04, S_{x3} = 0.015, n = 0.016, Q = 2.0 ft^3/s, shoulder width = 6 ft).

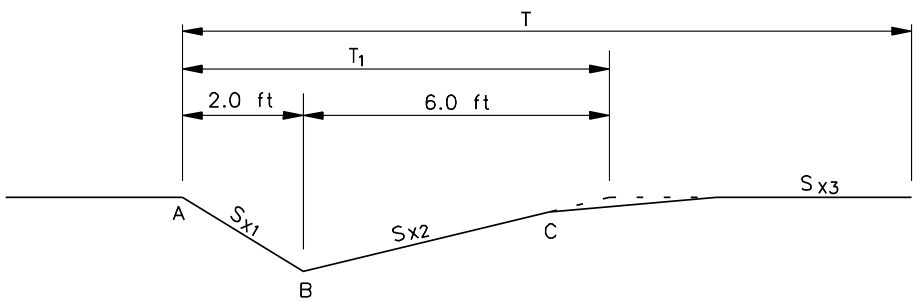


FIGURE 13-4 — V-Type Gutter

Step 2 Calculate S_x:

$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$ $S_x = (0.06)(0.04)/(0.06 + 0.04) = 0.024$

Step 3 Solve for T_1 using the nomograph on Figure 13-1:

T_1 is a hypothetical width that is correct if it is contained within S_{x1} and S_{x2}. From the nomograph, T_1 = 8.4 ft; however, because the shoulder width of 6 ft is less than 8.4 ft, S_{x2} is 0.04, and the pavement cross slope S_{x3} is 0.015. T will actually be greater than 8.4 ft, 8.4 – 2.0 = 6.4 ft, which is > 4.0 ft; therefore, the spread is greater than 8.4 ft.

Step 4 To find the actual spread, solve for depth at Points B and C:

Point B: 6.4 ft @ 0.04 = 0.26 ft
Point C: 0.26 ft – (4.0 ft @ 0.04) = 0.1 ft

Step 5 Solve for the spread on the pavement. (Pavement cross slope = 0.015):

$T_{0.015} = 0.1/0.015 = 6.7$ ft

Step 6 Find the actual total spread (T): T = 6.0 + 6.7 = 12.7 ft

CONDITION 2: Given spread (T), find flow (Q):

Step 1 Determine input parameters such as longitudinal slope (S), cross slope ($S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2})$), Manning's n and allowable spread. (Example: n = 0.016, S = 0.015, S_{x1} = 0.06, S_{x2} = 0.04, T = 6.0 ft).

Step 2 Calculate S_x:

$$S_x = S_{x1}S_{x2}/(S_{x1} + S_{x2}) = (0.06)(0.04)/(0.06 + 0.04) = 0.024$$

Step 3 Using Figure 13-1, solve for Q: For T = 6.0 ft, Q = 1.1 ft^3/s

The Equation shown on Figure 13-1 can also be used.

13.11 INLETS

13.11.1 <u>General</u>

Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm drains or to culverts. Grate inlets should be bicycle safe, unless located on highways where bicycles (or wheelchair traffic) are not permitted.

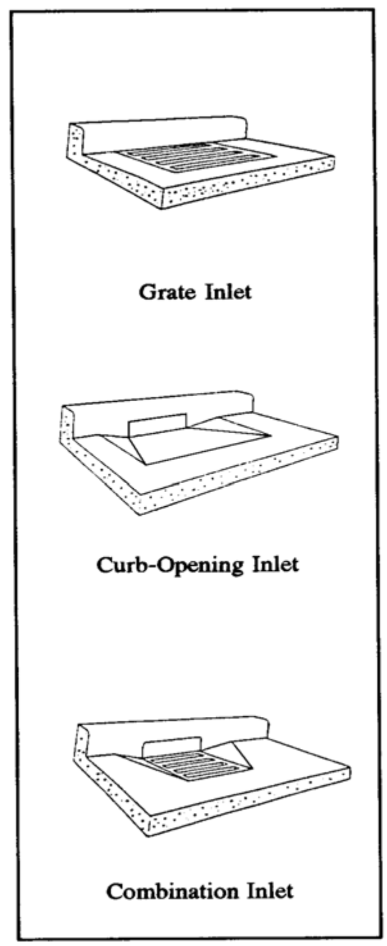
This Section discusses the various types of inlets in use and recommends guidelines on the use of each type.

13.11.2 <u>Types</u>

Inlets used for the drainage of highway surfaces can be divided into four major classes. These classes are as follows.

13.11.2.1 Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. They are best suited for use on continuous grades. Because they are susceptible to clogging with debris, the use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. Special-design (oversize) grate inlets can be utilized at major sag points if sufficient capacity is provided for clogging. In this case, flanking inlets are definitely recommended. Grates should be bicycle safe where bike traffic is anticipated and structurally designed to handle the appropriate loads where subject to traffic. See Sections 13.12.3 and 13.12.8 for additional discussion.



Inlet Types

13.11.2.2 Curb-Opening Inlets

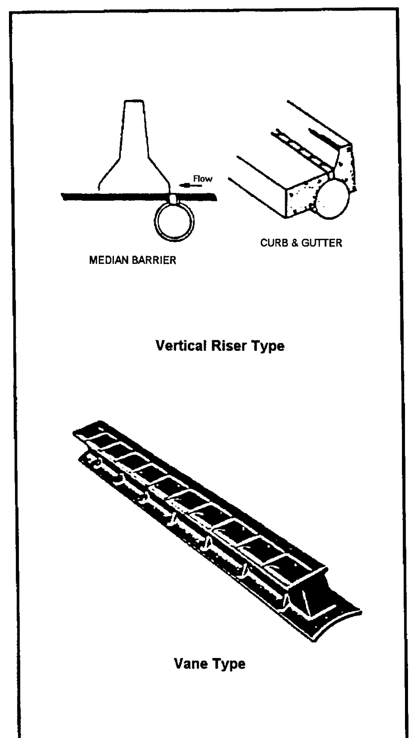
These inlets are vertical openings in the curb covered by a top slab. They are best suited for use at sag points because they can convey large quantities of water and debris. They are a viable alternative to grates in many locations where grates would be hazardous for pedestrians or bicyclists. They are generally not recommended for use on steep continuous grades.

13.11.2.3 Combination Inlets

Various types of combination inlets are in use. Curb-opening and grate combinations are common — some with the curb opening upstream of the grate, and some with the curb opening adjacent to the grate. Slotted inlets are also used in combination with grates, located either longitudinally upstream of the grate or transversely adjacent to the grate. Engineering judgment is necessary to determine if the total capacity of the inlet is the sum of the individual components or a portion of each. The gutter grade, cross slope and proximity of the inlets to each other will be deciding factors. Combination inlets may be desirable in sags because they can provide additional capacity in the event of plugging.

13.11.2.4 Slotted Drain/Trench Drain Inlets

These inlets consist of a slotted opening with bars perpendicular to the opening. Slotted inlets function as weirs with flow entering from the side. They can be used to intercept sheet flow, collect gutter flow with or without curbs, modify existing systems to accommodate roadway widening or increased runoff, and reduce ponding depth and spread at grate inlets. The two types of slotted inlets in general use are the vertical riser type and the vane type.



Slotted Inlets

13.11.3 Inlet Locations

Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria (see Section 13.9). In addition, there are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity or runby. Examples of such locations are as follows:

- sag points in the gutter grade;
- upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections;
- immediately upstream and downstream of bridges;
- immediately upstream of cross slope reversals;
- on side streets at intersections;
- at the end of channels in cut sections;
- behind curbs, shoulders or sidewalks to drain low areas; and
- where necessary to collect snow melt.

Inlets should not be located in the path where pedestrians are likely to walk.

13.12 INLET SPACING

13.12.1 General

A number of inlets are required to collect runoff at locations with little regard for contributing drainage area as discussed in Section 13.11.3. These should be plotted on the plan first. Next, it is recommended to start locating inlets from the crest and working downgrade to the sag points. The location of the first inlet from the crest can be found by determining the length of pavement and the area back of the curb sloping toward the roadway that will generate the design runoff. The design runoff can be computed as the maximum allowable flow in the curbed channel that will meet the design criteria as specified in Section 13.9. Where the contributing drainage area consists of a strip of land parallel to and including a portion of the highway, the first inlet can be calculated as follows:

$$L = \frac{43{,}560\ Q_t}{CIW} \tag{13.7}$$

where:
- L = distance from the crest, ft
- Q_t = maximum allowable flow, ft^3/s
- C = composite runoff coefficient for contributing drainage area
- W = width of contributing drainage area, ft
- I = rainfall intensity for design frequency, in/h

If the drainage area contributing to the first inlet from the crest is irregular in shape, trial and error will be necessary to match a design flow with the maximum allowable flow. Equation 13.7 is an alternative form of the Rational Equation.

To space successive downgrade inlets, it is necessary to compute the amount of flow that will be intercepted by the inlet (Q_i) and subtract it from the total gutter flow to compute the runby. The runby from the first inlet is added to the computed flow to the second inlet, the total of which must be less than the maximum allowable flow dictated by the criteria. Table 13-4 (Section 13.12.9) is an inlet spacing computation sheet that can be utilized to record the spacing calculations.

Inlet interception capacity of all types of grate inlets slotted drain inlets, and curb inlets have been investigated by FHWA. References *(9)*, *(6)* may be used to analyze the flow in gutters and the interception capacity of all types of inlets on continuous grades and sags. Both uniform and composite cross slopes can be analyzed.

13.12.2 Grate Inlets On Grade

The capacity of a grate inlet depends upon its geometry, cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb-opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, a portion of the frontal flow may tend to

splash over the end of the grate for some grates. Figure 13-5 can be utilized to determine splash-over velocities for various grate configurations and the portion of frontal flow intercepted by the grate. Note that the parallel bar grates are the most efficient grates on steep slopes but are not bicycle safe. The grates tested in a FHWA research study are described in HEC 22 *(9)*.

The ratio of frontal flow to total gutter flow, E_o, for a straight cross slope is given by the following Equation:

$$E_o = Q_w/Q = 1 - (1 - W/T)^{2.67} \qquad (13.8)$$

where:
- Q = total gutter flow, ft^3/s
- Q_w = flow in width W, ft^3/s
- W = width of depressed gutter or grate, ft
- T = total spread of water in the gutter, ft

Figure 13-2 provides a graphical solution of E_o for either straight cross slopes or depressed gutter sections.

The ratio of side flow, Q_s, to total gutter flow is:

$$Q_s/Q = 1 - Q_w/Q = 1 - E_o \qquad (13.9)$$

The ratio of frontal flow intercepted to total frontal flow, R_f, is expressed by the following Equation:

$$R_f = 1 - 0.09\,(V - V_o) \qquad (13.10)$$

where:
- V = velocity of flow in the gutter, ft/s
- V_o = gutter velocity where splash-over first occurs, ft/s

This ratio is equivalent to frontal-flow interception efficiency. Figure 13-5 provides a solution of Equation 13.10 that incorporates grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 13-5 is total gutter flow divided by the area of flow.

Figure 13-6 is a nomograph to solve for velocity in a triangular gutter section with known cross slope, slope and spread.

The ratio of side flow intercepted to total side flow, R_s, or side-flow interception efficiency, is expressed by:

$$R_s = 1 / [1 + (0.15V^{1.8}/S_xL^{2.3})] \qquad (13.11)$$

where:
- V = velocity of flow in gutter, ft/s
- L = length of the grate, ft
- S_x = cross slope, ft/ft

Figure 13-7 provides a solution to Equation 13.11.

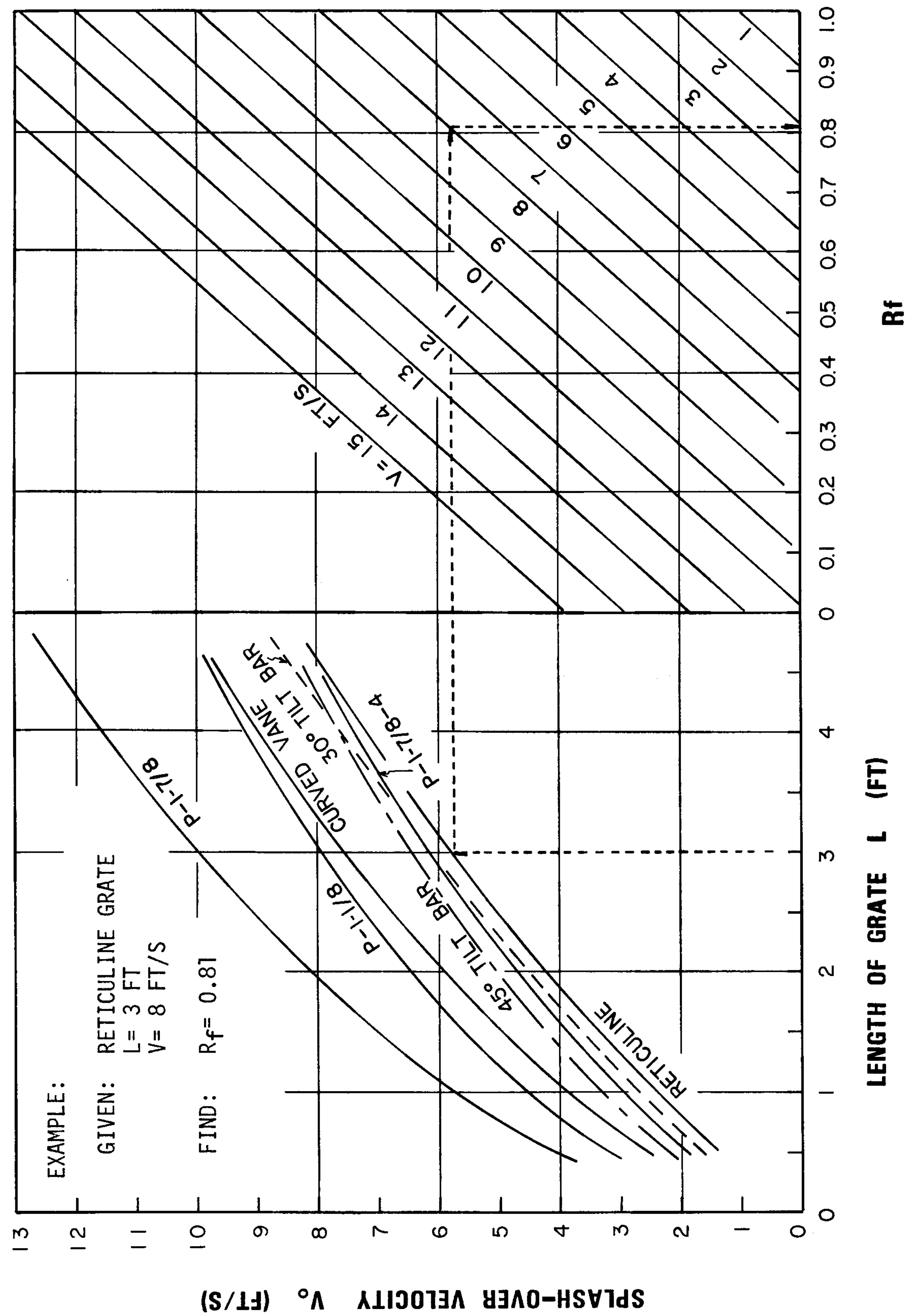
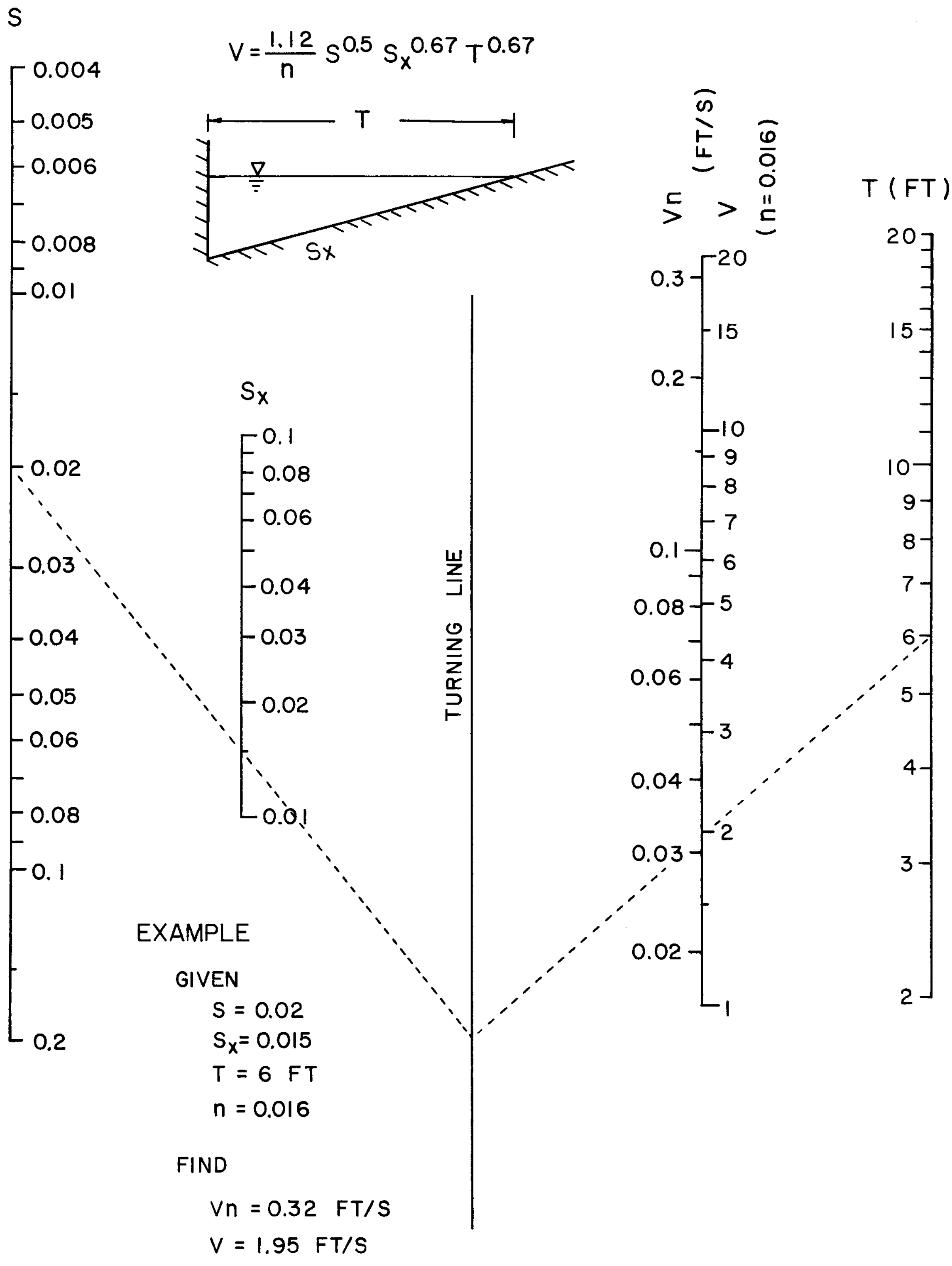


FIGURE 13-5 — Grate Inlet Frontal-Flow Interception Efficiency

Source: HEC 22 (9).



Velocity in Triangular Gutter Sections - English Units

FIGURE 13-6 — Velocity in Triangular Gutter Sections

Source: HEC 22 *(9)*.

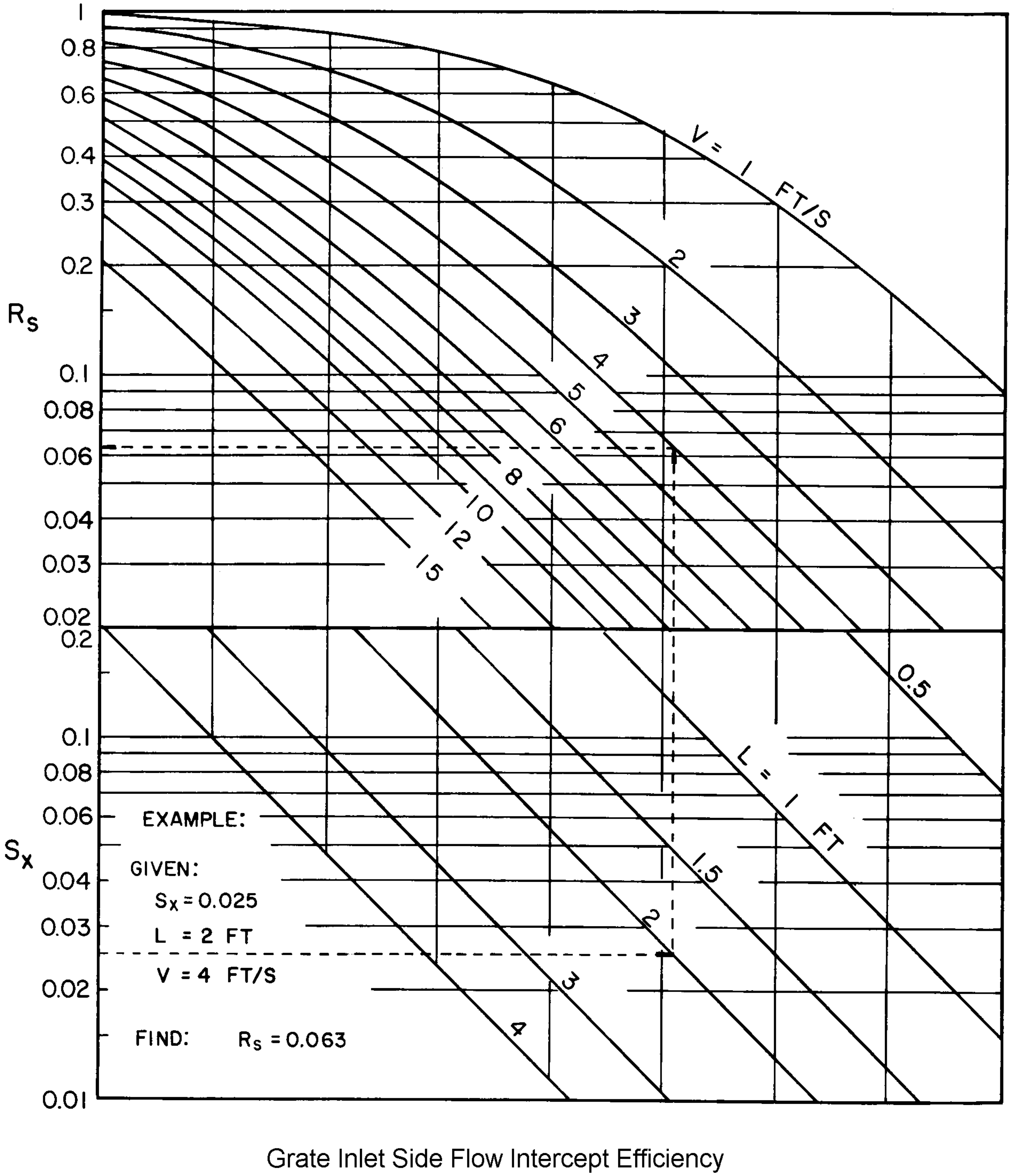


FIGURE 13-7 — Grate Inlet Side-Flow Interception Efficiency

Source: HEC 22 *(9)*.

The efficiency, E, of a grate is expressed as:

$$E = R_f E_o + R_s(1 - E_o) \qquad (13.12)$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

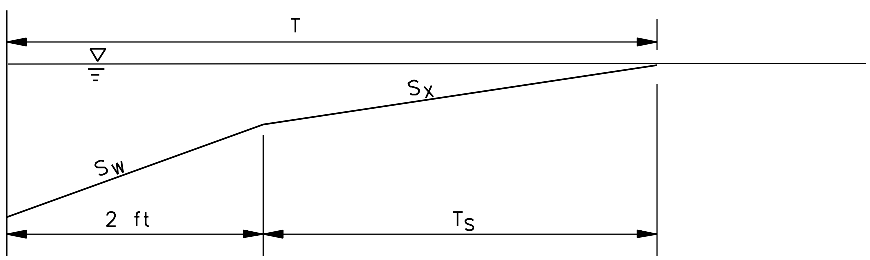
$$Q_i = EQ = Q[R_f E_o + R_s(1 - E_o)] \qquad (13.13)$$

Example Problem

Given: Drainage area: 200-ft residential strip, C = 0.4, S = 0.05
12-ft lane @ 0.015, 8-ft shoulder at 0.04, and 2-ft gutter at 0.06
10-yr design, IDF Curve (Figure 7-5)
Allowable spread, T = 2 ft, n = 0.016
$S_o = 0.01$, $S_x = 0.04$, $S_w = 0.06$
Use curves & nomographs in *MDM*

Find: Maximum allowable flow, Q_T
Q_i intercepted by 2-ft × 2-ft vane grate
Q_r runby
Location of first and second inlets from crest of hill

Sketch:



Solution:

1. Solve for Q_s using Figure 13-1:

   $T_s = 8$ ft; $S_x = 0.04$; $Q_s = 3.9$ ft^3/s

2. Use Figure 13-2 to find E_o:

   $S_w/S_x = 0.06/0.04 = 1.5$ W/T = 0.6/3.0 = 0.2 $E_o = 0.47 = Q_w/Q$

3. Find total Q_T (maximum allowable flow):

   $Q_T = Q_s/(1 - E_o) = 3.9/(1 - 0.47) = \underline{\underline{7.4 \text{ ft}^3/\text{s}}}$

4. From Figure 13-6, V = 3.9 ft/s.

5. From Figure 13-5, $R_f = 1.0$; from Figure 13-7, $R_s = 0.1$.

6. Using Equation 13.13:

   $Q_i = Q_T\,[R_f E_o + R_s\,(1 - E_o)]$
   $Q_i = 7.4[(1.0)(0.47) + 0.1(1 - 0.47)] = \underline{\underline{3.9\ ft^3/s}}$

7. $Q_r = Q_T - Q_i$ $\quad Q_r = 7.4 - 3.9 = \underline{\underline{3.5\ ft^3/s}}$

8. Locate first inlet from crest. Using Equation 13.7:

$$L = \frac{43{,}560\ Q_t}{CIW} \tag{13.14}$$

   where:
   - L = distance from the crest, ft
   - Q_t = maximum allowable flow, ft^3/s
   - C = composite runoff coefficient for contributing drainage area
   - W = width of contributing drainage area, ft
   - I = rainfall intensity for design frequency, in/h

   To find I, first solve for t_c; use Equation 7.12 for grassed waterway (shallow concentrated flow)
   C = 0.4, S = 0.5%; From Table 7-6, K = 1.499
   $V = (1.499)\ (0.5)^{0.5} = 1.1$ ft/s; $t_t = 200/((1.1)(60)) = 3$ min
   Gutter flow estimated at V = 3.9 ft/s (from Figure 13-6)
   Try 440 ft; $t_c = 440/(3.9 \times 60) = 2$ min; Total $t_c = 2 + 3 = 5$ min
   From Figure 7-5, I = 6.1 in/h
   Solve for weighted C value: C= [(200)(0.4) + (24)(0.9)]/224 = 0.454
   L = 43,560(7.4)/((0.454)(6.1)(224)) = 520 ft OK

   <u>Place first inlet 520 ft from crest.</u>

9. To locate second inlet:

   $Q_T = 7.4\ ft^3/s$, $Q_r = 3.5\ ft^3/s$, $Q_{allowable} = 7.4 - 3.5 = 3.9\ ft^3/s$
   Assuming similar drainage area and t_c, I = 6.1 in/h
   L = 43,560(3.9)/((0.454)(6.1)(224)) = 274 ft

   <u>Place second inlet 274 ft from first inlet.</u>

13.12.3 <u>Grate Inlets In Sag</u>

Although curb-opening inlets are generally preferred to grate inlets at a sag, grate inlets can be used successfully. For minor sag points where debris potential is limited, grate inlets without a curb-opening inlet can be utilized. An example of a minor sag point might be on a ramp as it joins a mainline. Curb-opening inlets in addition to a grate are preferred at sag points where debris is likely, such as on a city street. For major sag points, such as on divided high-speed highways, a curb-opening inlet is preferable to a grate inlet because of its hydraulic capacity and debris-handling capabilities. When grates are used, it is good practice to assume half the grate is clogged with debris.

Where significant ponding can occur, in locations such as underpasses and in sag vertical curves in depressed sections, it is good engineering practice to place a minimum of one flanking inlet on each side of the sag point inlet. The flanking inlets should be placed so that they will limit spread on low-gradient approaches to the low point and act in relief of the inlet at the low point if it should become clogged or if the allowable spread is exceeded. A further discussion and methodology is given in Section 13.12.8.

A grate inlet in a sag operates as a weir up to a depth of about 0.4 ft and as an orifice for depths greater than 1.4 ft. Between these depths, a transition from weir to orifice flow occurs. The capacity of a grate inlet operating as a weir is:

$$Q_i = CPd^{1.5} \tag{13.15}$$

where:
- P = perimeter of grate excluding bar widths and side against curb, ft
- C = 3.0, weir coefficient
- d = flow depth, ft

The capacity of a grate inlet operating as an orifice is:

$$Q_i = CA(2gd)^{0.5} \tag{13.16}$$

where:
- C = 0.67, orifice coefficient
- A = clear opening area of the grate, ft^2
- g = 32.2 ft/s^2

Figure 13-8 is a plot of Equations 13.15 and 13.16 for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

Example Problem

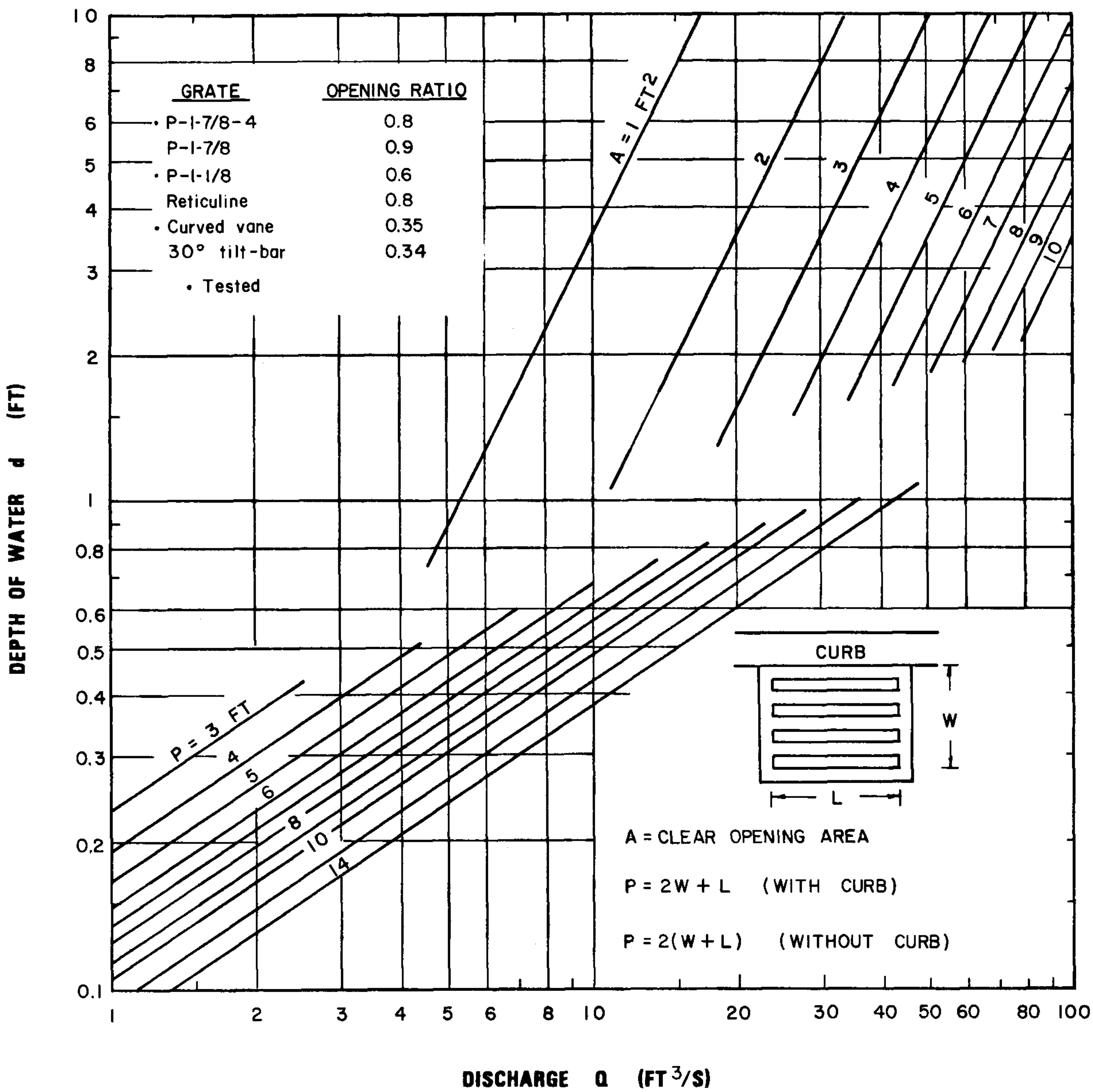
The following Example illustrates the use of Figure 13-8:

Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate:

$Q_r = 3.5\ ft^3/s$	$Q = 8\ ft^3/s$, design storm
$Q_r = 4.2\ ft^3/s$	$Q = 11\ ft^3/s$, check storm
$S_x = 0.05$ ft/ft	T = 10 ft design
$d = TS_x = 0.5$ ft	n = 0.016

Find: Grate size for design Q and depth at curb for check Q. Check spread at S = 0.003 on approaches to the low point.

Solution: From Figure 13-8, a grate must have a perimeter of 8 ft to intercept 8 ft^3/s at a depth of 0.5 ft. Some assumptions must be made regarding the nature of the clogging to compute the capacity of a partially clogged grate. If the area of a grate is 50%



Grate Inlet Capacity in Sump Conditions - English Units

FIGURE 13-8 — Grate Inlet Capacity in Sump Conditions

Source: HEC 22 *(9)*.

covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50%. For example, if a 2-ft x 4-ft grate is clogged so that the effective width is 1 ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50% and the perimeter by 25%. Therefore, assuming 50% clogging along the length of the grate, a 4-ft × 4-ft, a 2-ft × 6-ft, or a 3-ft × 5-ft grate would meet requirements of an 8-ft perimeter 50% clogged.

Assuming that the installation chosen to meet design conditions is a double 2-ft × 3-ft grate, for 50% clogged conditions:

P = 1 + 6 + 1 = 8 ft

For design flow: d = 0.5 ft (from Figure 13-8)

For check flow: d = 0.6 ft (from Figure 13-8); T = 12 ft

At the check flow rate, ponding will extend 2 ft into a traffic lane if the grate is 50% clogged in the manner assumed.

AASHTO geometric policy recommends a gradient of 0.3% within 50 ft of the level point in a sag vertical curve. Check T at S = 0.003 for the design and check flow:

Q = 3.5 ft^3/s, T = 8.2 ft (design storm) (Figure 13-1)
Q = 4.2 ft^3/s, T = 9 ft (check storm) (Figure 13-1)

Thus, a double 2-ft × 3-ft grate 50% clogged is adequate to intercept the design flow at a spread that does not exceed design spread, and the spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in a sag where ponding can occur, and flanking inlets on the low-gradient approaches.

13.12.4 Curb Inlets on Grade

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of a curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}(1/nS_x)^{0.6} \qquad (13.17)$$

where: K = 0.6
L_T = curb-opening length required to intercept 100% of the gutter flow, ft

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - (1 - L/L_T)^{1.8} \qquad (13.18)$$

where: L = curb-opening length, ft

Figure 13-9 is a nomograph for the solution of Equation 13.17, and Figure 13-10 provides a solution of Equation 13.18.

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in Equation 13.17:

$$S_e = S_x + S'_w E_o \qquad (13.19)$$

where: S'_w = cross slope of the gutter measured from the cross slope of the pavement
= (a/12W), ft/ft

a = gutter depression, in

E_o = ratio of flow in the depressed section to total gutter flow. It is determined by the gutter configuration upstream of the inlet. Reference Figure 13-2 to determine E_o.

Note: S_e can be used to calculate the length of curb opening by substituting S_e for S_x in Equation 13.17.

Example Problem

The following Example illustrates the use of this procedure:

Given: S_x = 0.03 ft/ft S = 0.035 ft/ft n = 0.016 Q = 5 ft^3/s

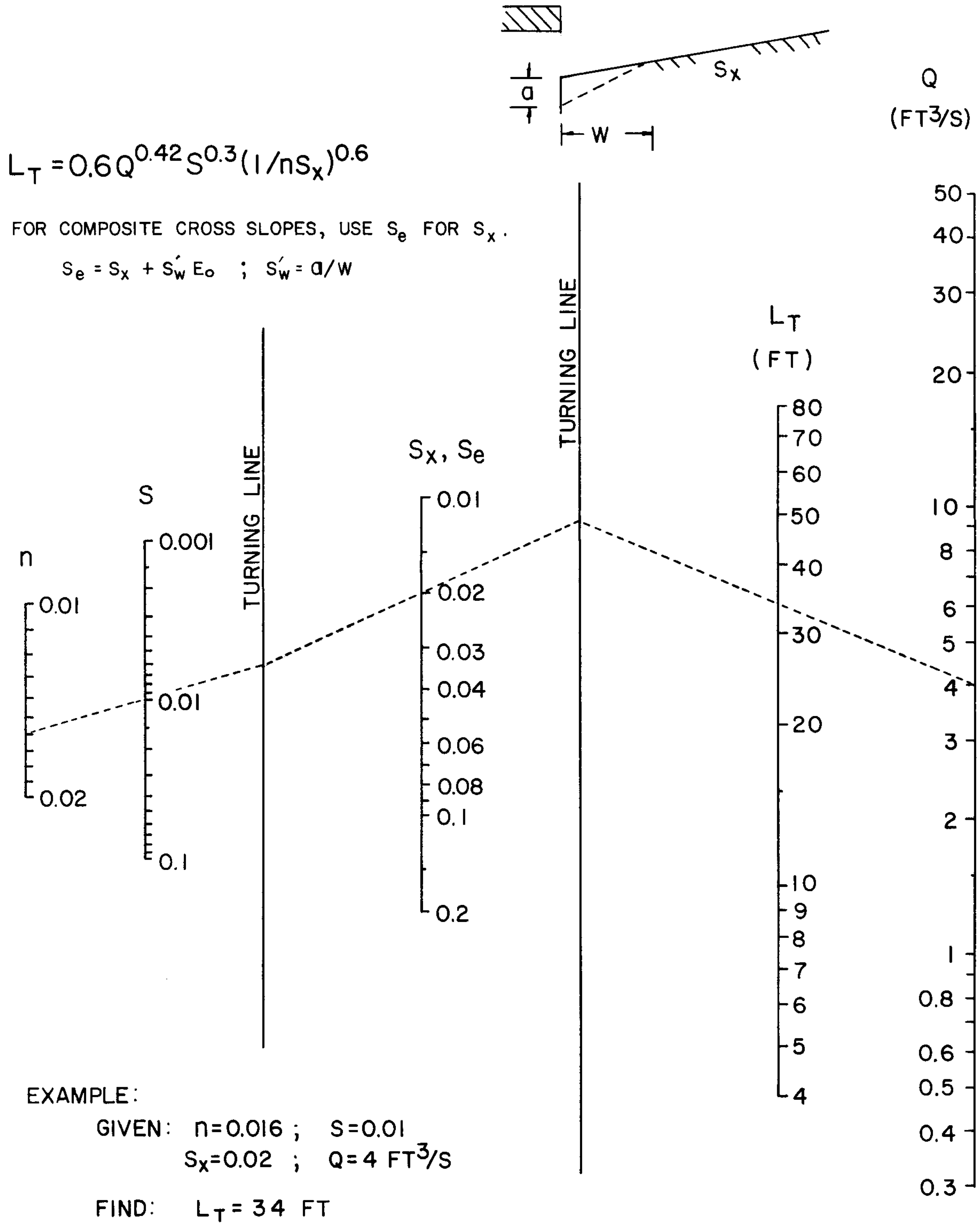
Find: (1) Q_i for 10-ft curb-opening inlet, uniform cross slope

(2) Q_i for a depressed 10-ft curb-opening inlet with composite cross slope
a = 2 in, W = 2 ft

(3) Q_i for a depressed 10-ft curb-opeing inlet with uniform cross slope

Solution: (1) From Figure 13-1, T = 8 ft
From Figure 13-9, L_T = 41 ft
L/L_T = 10/41 = 0.24
From Figure 13-10, E = 0.39
Q_i = EQ = (0.39)(5) = $\underline{\underline{2\ ft^3/s}}$

(2) Qn = (5)(0.016) = 0.08 ft^3/s
S_w/S_x = (0.03 + 0.085)/0.03 = 3.83
From Figure 13-3, T/W = 3.5 and T = 7 ft



Curb-opening & Slotted Drain Inlet Length for Total Interception - English Units

FIGURE 13-9 — Curb-Opening and Longitudinal Slotted Drain Inlet Length For Total Interception

Source: HEC 22 *(9).*

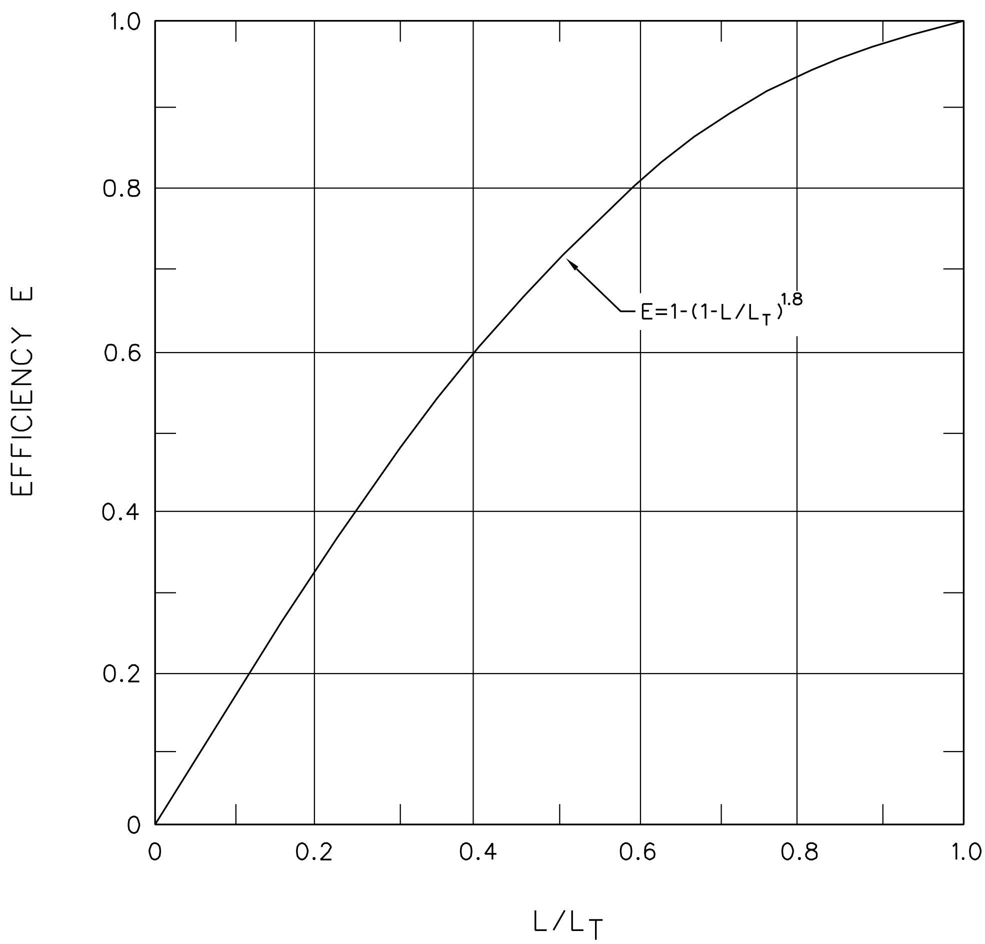


FIGURE 13-10 — Curb-Opening and Slotted Drain Inlet Interception Efficiency

Source: HEC 22 *(9).*

Then W/T (Depress) = 2/7 = 0.29
From Figure 13-2, E_o = 0.74
$S_e = S_x + S'_w E_o = 0.03 + 0.085(0.74) = 0.09$
From Figure 13-9, L_T = 21 ft, then L/L_T = 10/21 = 0.48
From Figure 13-10, E = 0.69, then Q_i = (0.69)(5) = <u>3.5 ft^3/s</u>

(3) S_w / S_x = 0.03 /0.03 = 1
W/T = 2/8 = 0.25
From Figure 13-2, E_o = 0.53
S_e = 0.03 + (0.085)(0.53) = 0.075
From Figure 13-9, L_T = 25 ft, then L/L_T = 10/25 = 0.4
From Figure 13-10, E = 0.60, then Q_i = (0.6)(5) = <u>3 ft^3/s</u>

13.12.5 Curb Inlets in Sag

The capacity of a curb-opening inlet in a sag depends on water depth at the curb, the curb-opening length and the height of the curb opening. The inlet operates as a weir to depths equal to the curb-opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The Equation for the interception capacity of a depressed curb-opening inlet operating as a weir is:

$$Q_i = C_W(L + 1.8W)d^{1.5} \qquad (13.20)$$

where:
C_W = 2.3
L = length of curb opening, ft
W = width of depression, ft
d = depth of water at curb measured from the normal cross slope gutter flow line, ft

See Figure 13-11 for a definition sketch.

The weir equation for curb-opening inlets without depression becomes:

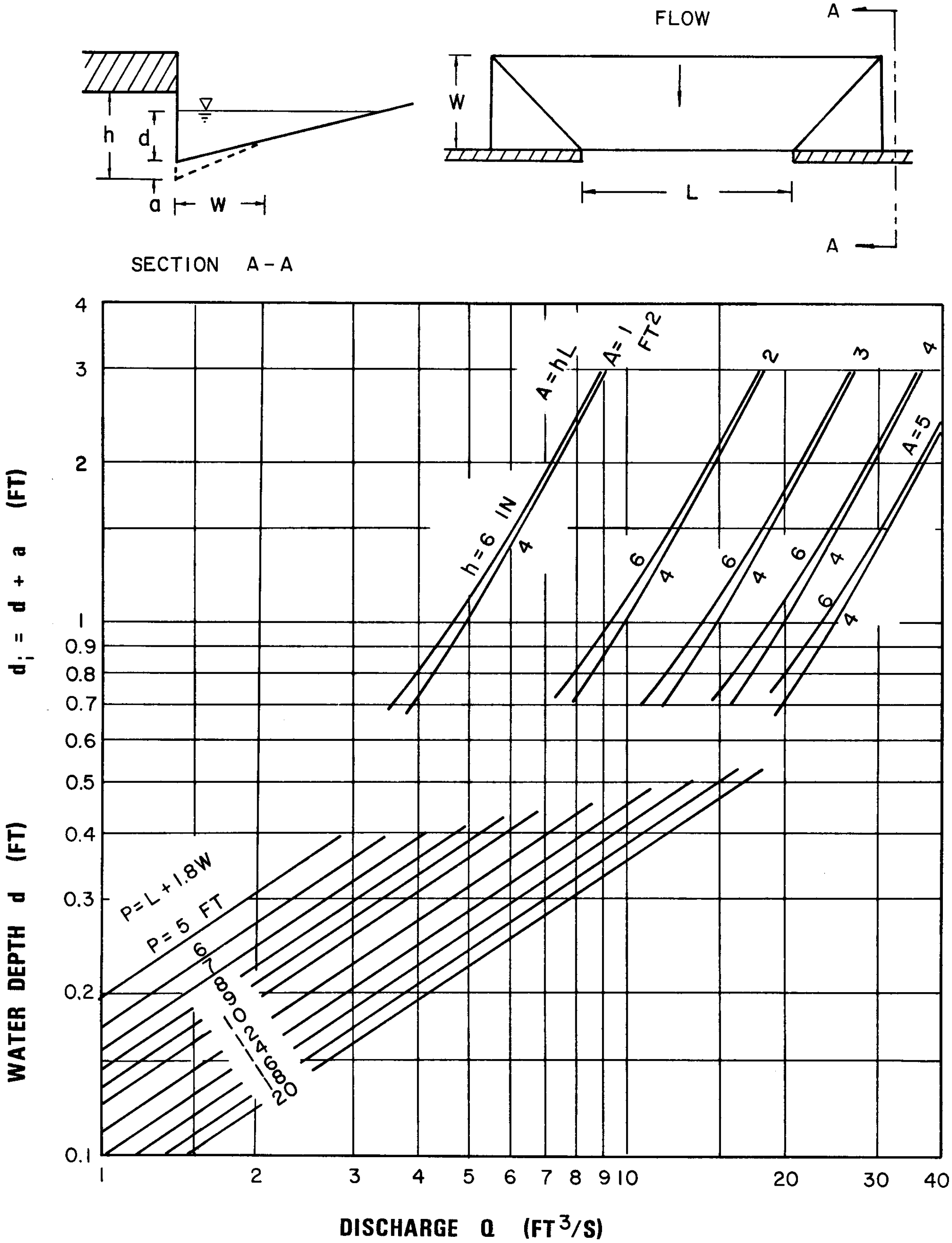
$$Q_i = C_W L d^{1.5} \qquad (13.21)$$

The depth limitation for operation as a weir becomes: $d \le h$.

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 × height of curb opening. The interception capacity can be computed by:

$$Q_i = C_o A[2g(d_i - h/2)]^{0.5} \qquad (13.22)$$

where:
C_o = orifice coefficient (0.67)
h = height of curb-opening orifice, ft
A = clear area of opening, ft^2
d_i = depth at lip of curb opening, ft



Depressed Curb-opening Inlet Capacity in Sump Locations - English Units

FIGURE 13-11 — Depressed Curb-Opening Inlet Capacity in Sump Locations

Source: HEC 22 *(9)*.

The weir equations use an effective weir length and coefficient that is representative of the line of gutter transition to the depression. The user should be cautioned not to use the depth from the water surface to the depressed inlet throat, but to the undepressed depth (or more specifically, the depth at the beginning of the transition). Otherwise, the capacity for weir flow will be overestimated.

Note: Equation 13.22 is applicable to depressed and undepressed curb-opening inlets, and the depth at the inlet includes any gutter depression.

Example Problem

The following Example illustrates the use of this procedure:

Given: Curb-opening inlet in a sump location:

$L = 5$ ft $h = 5$ in

(1) Undepressed curb opening:

$S_x = 0.05$ $T = 8$ ft

(2) Depressed curb opening:

$S_x = 0.05$ $W = 2$ ft
$a = 2$ in $T = 8$ ft

Find: Q_i

Solution: (1) $d = TS_s = (8)(0.05) = 0.4$ ft d < h; therefore, weir controls
$Q_i = C_W L d^{1.5} = (2.3)(5)(0.4)^{1.5} = 2.9$ ft^3/s

2) $d = 0.4$ ft $< (1.4\ h) = 0.6$; therefore, weir controls
$P = L + 1.8W = 5 + 1.8(2) = 8.6$ ft
$Q_i = (2.3)(8.6)(0.4)^{1.5} = 5$ ft^3/s (Figure 13-11)

At d = 0.4 ft, the depressed curb-opening inlet has about 70% more capacity than an inlet without depression. In practice, the flow rate would be known and the depth at the curb would be unknown.

13.12.6 Slotted Inlets on Grade

Slotted inlets are effective pavement drainage inlets that have a variety of applications. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. They can be placed longitudinally in the gutter or transversely to the gutter. Slotted inlets should generally be connected into inlet structures so that they will be accessible to maintenance forces in case of plugging or freezing.

13.12.6.1 Longitudinal Placement

Flow interception by slotted inlets and curb-opening inlets is similar in that each is a side weir, and the flow is subjected to lateral acceleration due to the cross slope of the pavement. Slotted inlets may have economic advantages in some cases and could be very useful on widening and safety projects where right-of-way is narrow and existing inlet capacity must be supplemented. In some cases, curbs can be eliminated as a result of utilizing slotted inlets.

The length of a slotted inlet required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = KQ^{0.42}S^{0.3}(1/nS_x)^{0.6} \qquad (13.23)$$

where: K = 0.6
L_T = slotted inlet length required to intercept 100% of the gutter flow, ft

The slot width must be at least 1¾ in for Equation 13.23 to be valid.

The efficiency of slotted inlets shorter than the length required for total interception is expressed by:

$$E = 1 - (1 - L/L_T)^{1.8} \qquad (13.24)$$

where: L = slotted inlet length, ft

Figure 13-9 is a nomograph for the solution of Equation 13.23, and Figure 13-10 provides a solution of Equation 13.24.

The length of inlet required for total interception by a slotted inlet in a composite section can be found by the use of an equivalent cross slope, S_e, in Equation 13.23:

$$S_e = S_x + S'_w E_o \qquad (13.25)$$

where: S_x = pavement cross slope, ft/ft
S_w = gutter cross slope, ft/ft
S'_w = $S_w - S_x$
E_o = ratio of flow in the depressed gutter to total gutter flow, Q_w/Q (see Figure 13-2)

Note that the same equations are used for both slotted inlets and curb-opening inlets. The following Example illustrates the use of this procedure:

<u>Example Problem</u>

Given: Longitudinal placement of slotted inlet adjacent to curb:

S_o = 0.01 ft/ft   Allowable spread = 10 ft n = 0.016

(1) Uniform cross slope, S_x = 0.02
(2) Composite cross slope, S_x = 0.02, S_w = 0.06
(3) Increase S_o to 0.03 and solve for (1) and (2)

Find: (1) Maximum allowable Q.
Q_i for a 10-ft slotted inlet on straight cross slope.

(2) Maximum allowable Q.
Q_i for a 10-ft slotted inlet on composite cross slope.

(3) Same as above with profile grade increased to 0.03 ft/ft.

Solution: (1) For T = 10 ft, Max Q = 2.4 ft^3/s from Figure 13-1:

L_T = 27 ft from Figure 13-9 $L/L_T = 10/27 = 0.37$
E = 0.55 from Figure 13-10; $Q_i = EQ = (0.55)(2.4) =$ 1.32 ft^3/s int.

(2) $Q_s = 1.32\ ft^3/s$ from Figure 13-1; W/T = 2/10 = 0.2:

$S_w/S_x = 0.06/0.02 = 3$; $E_o = 0.52$ from Figure 13-2
Max $Q = Q_s/(1 - E_o) = 1.32/(1 - 0.52) =$ 2.8 ft^3/s
$S'_w = S_w - S_x = 0.06 - 0.02 = 0.04$
$S_e = S_x + S'_w E_o = 0.02 + (0.04)(0.52) = 0.041$
L_T = 18 ft from Figure 13-9; $L/L_T = 10/18 = 0.56$ E = 0.76 from Figure 13-10; then $Q_i = EQ = (0.76)(2.8) =$ 2.1 ft^3/s intercepted

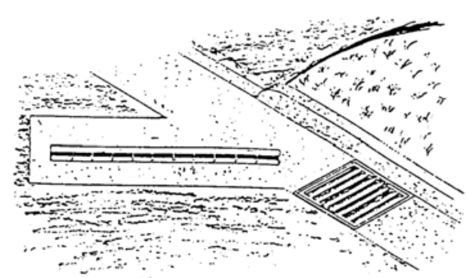
The slotted inlet in the composite gutter section will intercept 1.6 times the flow intercepted by the slotted inlet in the uniform section.

(3) From a similar analysis for $S_o = 0.03$:

Uniform Section: Max Q = 4.1 ft^3/s and $Q_i = 1.4\ ft^3/s$
Composite Section: Max Q = 4.6 ft^3/s and $Q_i = 2.3\ ft^3/s$

13.12.6.2 Transverse Placement

At locations where it is desirable to capture virtually all of the flow in the curbed section, a slotted vane drain can be installed in conjunction with a grate inlet. Tests have indicated that, when the slotted vane drain is installed perpendicular to the flow, it will capture approximately 0.5 ft^3/s per lineal meter of drain on longitudinal slopes of 0% to 6%. Capacity curves are available from the manufacturer. The ideal installation would utilize a grate inlet to capture the flow in the gutter and the slotted vane drain to collect the flow extending into the shoulder. Note that a slotted vane drain is shaped and rounded to increase inlet efficiency and should not be confused with a standard vertical riser type slotted inlet.



Slotted Vane Drain

13.12.7 Slotted/Trench drain Inlets In Sag

The use of slotted drain inlets in sag configurations is generally discouraged because of the propensity of such inlets to intercept debris in sags. However, there may be locations where it is desirable to supplement an existing low-point inlet with the use of a slotted drain. Slotted inlets in sag locations perform as weirs to depths of about 0.2 ft, dependent on slot width and length. At depths greater than about 0.4 ft, they perform as orifices. Between these depths, flow is in a transition stage. The interception capacity of a slotted inlet operating as an orifice can be computed by the following Equation:

$$Q_i = 0.8LW(2gd)^{0.5} \quad (13.26)$$

where: W = width of slot, ft
L = length of slot, ft
d = depth of water at slot, ft
g = 32.2 ft/s²

For a slot width of 1¾ in, Equation 13.26 becomes:

$$Q_i = 0.94Ld^{0.5} \quad (13.26a)$$

The interception capacity of slotted inlets at depths between 0.2 ft and 0.4 ft can be computed by use of the orifice equation. The orifice coefficient varies with depth, slot width and the length of slotted inlet. Figure 13-12 provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

13.12.8 Flanking Inlets

At major sag points where significant ponding may occur, such as underpasses or sag vertical curves in depressed sections, it is recommended practice to place a minimum of one flanking inlet on each side of the inlet at the sag point. The flanking inlets should be placed to act in relief of the sag inlet if it should become clogged. Table 13-3 shows the spacing required for various depths-at-curb criteria and vertical curve lengths defined by K = L/A, where L is the length of the vertical curve in feet and A is the algebraic difference in approach grades. The AASHTO policy on geometrics specifies maximum K values for various design speeds and a maximum K of 167 considering drainage.

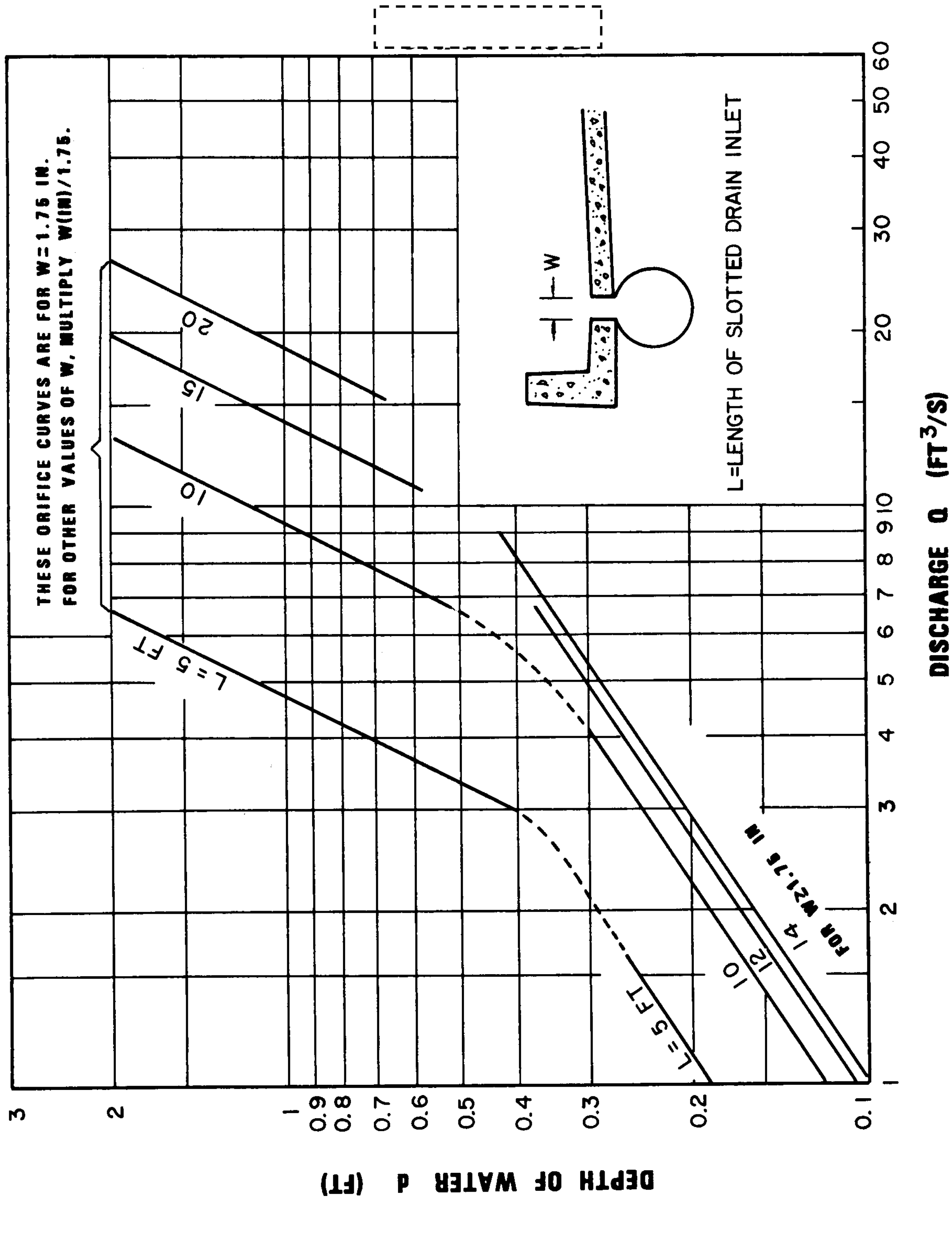
Example Problem

Given: V = 55 mph, K = 115 ft/%, S_x = 0.04, allowable spread (T) is 10 ft

Find: Location of flanking inlets that will function in relief of the inlet at the low point when the inlet at the low point is clogged.

Solution: (1) Determine depth at design speed: $d = S_x T = (0.04)(10) = 0.4$ ft

(2) Depth over flanking inlet to carry one-half of the design flow: d = 0.63 (0.4 ft) = 0.25 ft



Slotted Drain Inlet Capacity in Sump Locations - English Units

FIGURE 13-12 — Slotted Drain Inlet Capacity in Sump Locations

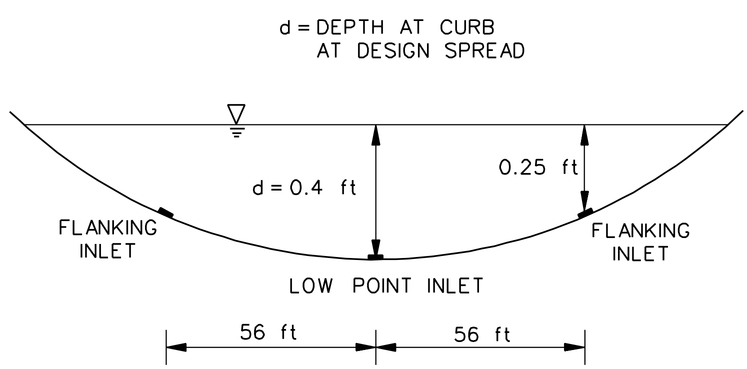
Source: HEC 22 (9).

TABLE 13-3 — Flanking Inlet Locations

x = Distance to flanking inlet in sag vertical curve locations using depth at curb criteria (ft).									
Speed (mph)	30	35	40	45	50	55	60	65	70
d↓ K→	37	49	64	79	96	115	136	157	181
0.1	27	31	36	40	44	48	52	56	60
0.2	38	44	51	56	62	68	74	79	85
0.3	47	54	62	69	76	83	90	97	104
0.4	54	63	72	79	88	96	104	112	120
0.5	61	70	80	89	98	107	117	125	135
0.6	67	77	88	97	107	117	128	137	147
0.7	72	83	95	105	116	127	138	148	159
0.8	77	89	101	112	124	136	148	158	170

NOTES:
1. $x = (200dK)^{0.5}$, where x = distance from the low point, ft.
2. Drainage maximum: K = 167 (ft/% A) for curbed facilities.
3. d = depth at curb, ft.
4. K = L/A where: L = Length of vertical curve, ft
   A = Algebraic difference in approach grades,%.
5. K-values in Table are based on providing stopping sight distances in Reference *(1)*.

EXAMPLE FLANKING INLETS AT SAG POINT



(3) Depth from bottom of sag to flanking inlet: 0.4 ft – 0.26 ft = 0.14 ft

(4) Spacing of flanking inlet = 56 ft (from Table 13-3, using d = 0.14 ft)

13.12.9 <u>Inlet Spacing Computations</u>

To design the location of the inlets for a given project, information such as a layout or plan sheet suitable for outlining drainage areas, road profiles, typical cross sections, grading cross sections, superelevation diagrams and contour maps are necessary. The inlet computation sheet, Table 13-4, should be used to document the computations. A step-by-step procedure is as follows:

Step 1 Complete the blanks on top of the sheet to identify the job by S.P., route, date and your initials.

Step 2 Mark on the plan the location of inlets that are necessary even without considering any specific drainage area. See Section 13.11.3 for additional information.

Step 3 Start at one end of the job, at one high point and work towards the low point, then space from the other high point back to the same low point.

Step 4 Select a trial drainage area approximately 300 ft to 500 ft below the high point, and outline the area including any area that may come over the curb. (Use drainage area maps). Where practical, large areas of behind-the-curb drainage should be intercepted before it reaches the highway. See Section 13.7.5.

Step 5 (Col 1) Describe the location of the proposed inlet by number and station in Columns 1 and 2.

(Col 5) Identify the curb and gutter type in the Remarks, Column 19. A sketch of the cross section should be provided in the open area of the computation sheet.

Step 6 (Col 3) Compute the drainage area in hectares and enter in Column 3.

Step 7 (Col 4) Select a C value from one of the tables in Chapter 7, Section 7.18, or compute a weighted value based on area and cover type as described in Section 13.6.2.1 and enter in Column 4.

Step 8 (Col 5) Compute a time of concentration for the first inlet. This will be the travel time from the hydraulically most remote point in the drainage area to the inlet. See additional discussion in Section 13.6.2.3 and in Chapter 7. The minimum time of concentration should be 7 min. Enter value in Column 5.

Step 9 (Col 6) Using the Intensity-Duration-Frequency curves, select a rainfall intensity at the t_c for the design frequency. Enter in Column 6.

Step 10 (Col 7) Calculate Q by multiplying Column 3 x Column 4 x Column 6. Enter in Column 7.

TABLE 13-4 — Inlet Spacing Computation Sheet

INLET COMPUTATION SHEET

DATE ____________ SP ____________ ROUTE ___________

COMPUTED BY __________ SHEET ___ OF ___

LOCATION		GUTTER DISCHARGE Design Frequency ______					GUTTER DISCHARGE Allowable Spread ______								INLET DISCHARGE			REMARKS
INLET NO.	STAT.	DRAIN AREA "A" (ac)	RUNOFF COEF "C"	TIME OF CONC. "t_C" (min)	Rain Intensity "I" (in/h)	Q= CIA/360 (ft^3/s)	GRADE "S_O" (ft/ft)	CROSS SLOPE "S_x" (ft/ft)	PREV RUNBY (ft^3/s)	TOTAL GUTTER FLOW (ft^3/s)	DEPTH "d" (ft)	GUTTER WIDTH "W" (ft)	SPREAD "T" (ft)	W/T	INLET TYPE	INTER-CEPT "Q_i" (ft^3/s)	RUNBY "Q_r" (ft^3/s)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19

Step 11 (Col 8)	Determine the gutter slope at the inlet from the profile grade — check effect of superelevation. Enter in Column 8.
Step 12 (Col 9) (Col 13)	Enter cross slope adjacent to inlet in Column 9 and gutter width in Column 13. Sketch composite cross slope with dimensions.
Step 13 (Col 11)	For the first inlet in a series (high point to low point), enter Column 7 in Column 11 since no previous runby has occurred yet.
Step 14 (Col 12) (Col 14)	Using Figure 13-1 or the available computer model, determine the spread T, enter in Column 14, and calculate the depth d at the curb by multiplying T times the cross slope(s) and enter in Column 12. Compare with the allowable spread as determined by the design criteria in Section 13.9. If Column 15 is less than the curb height and Column 14 is near the allowable spread, continue on to Step 16. If not OK, expand or decrease the drainage area to meet the criteria and repeat Steps 5 through 14. Continue these repetitions until Column 14 is near the allowable spread, then proceed to Step 15.
Step 15 (Col 15)	Calculate W/T and enter in Column 15.
Step 16 (Col 16)	Select the inlet type and dimensions and enter in Column 16.
Step 17 (Col 17)	Calculate the Q intercepted (Q_i) by the inlet and enter in Column 17. Use Figures 13-1 and 13-2 or 13-3 to define the flow in the gutter. Use Figures 13-2, 13-5 and 13-7 and Equation 13.13 to calculate Q_i for a grate inlet and Figures 13-9 and 13-10 to calculate Q_i for a curb-opening inlet. See Section 13.12.2 for a grate inlet example and Section 13.12.4 for a curb-opening inlet example.
Step 18 (Col 18)	Calculate the runby by subtracting Column 17 from Column 11, and enter into Column 18 and Column 10 on the next line if an additional inlet exists downstream.
Step 19 (Cols 1-4)	Proceed to the next inlet downgrade. Select an area approximately 300 ft to 400 ft below the first inlet as a first trial. Repeat Steps 5 through 7 considering only the area between the inlets.
Step 20 (Col 5)	Compute a time of concentration for the second inlet downgrade based on the area between the two inlets.
Step 21 (Col 6)	Determine the intensity based on the time of concentration determined in Step 19 and enter in Column 6.
Step 22 (Col 7)	Determine the discharge from this area by multiplying Column 3 x Column 4 x Column 6. Enter the discharge in Column 7.
Step 23 (Col 11)	Determine total gutter flow by adding Column 7 and Column 10 and enter in Column 11. Column 10 is the same as Column 18 from the previous line.

Step 24 (Col 12) (Col 14) Determine "T" based on total gutter flow (Column 11) by using Figure 13-1 or 13-3 and enter in Column 14. (If "T" in Column 14 exceeds the allowable spread, reduce the area and repeat Steps 19 through 24. If "T" in Column 14 is substantially less than the allowable spread, increase the area and repeat Steps 19 through 24).

Step 25 (Col 16) Select inlet type and dimensions and enter in Column 16.

Step 26 (Col 17) Determine Q_i and enter in Column 17 — see instruction in Step 17.

Step 27 (Col 18) Calculate the runby by subtracting Column 17 from Column 7 and enter in Column 16. This completes the spacing design for this inlet.

Step 28 Go back to Step 19 and repeat Step 19 through Step 27 for each subsequent inlet. If the drainage area and weighted "C" values are similar between each inlet, the values from the previous grate location can be reused. If they are significantly different, recomputation will be required.

13.13 ACCESS HOLES

13.13.1 Location

Access holes are used to provide entry to continuous underground storm drains for inspection and cleanout. Some agencies use grate inlets in lieu of access holes when entry to the system can be provided at the grate inlet, so that the benefit of extra stormwater interception can be achieved with minimal additional cost. Typical locations where access holes should be specified are:

- where two or more storm drains converge,
- at intermediate points along tangent sections,
- where pipe size changes,
- where an abrupt change in alignment occurs, and
- where an abrupt change of the grade occurs.

Access holes should not be located in traffic lanes; however, where it is impossible to avoid locating an access hole in a traffic lane, care should be taken to ensure that it is not in the normal vehicular wheel path.

13.13.2 Spacing

The spacing of access holes should be in accordance with the following criteria:

Size of Pipe (in)	Maximum Distance (ft)
18-24	300
27-36	400
42-54	500
≥ 60	1000

13.13.3 Types

Following are ranges of pipe diameters and respective access-holes inside diameter dimension.

- Where the storm drain pipe diameter is 18 in or less, a 48-in diameter access hole should be provided.
- Where the storm drain pipe diameter is 21 in to 42 in inclusive, a 60-in diameter access hole should be provided.
- Where the storm drain pipe diameter is 48 in or larger, a 72-in diameter access hole, a reinforced concrete pipe tee or a special-design access hole should be provided.

13.13.4 Sizing

When determining the minimum access hole size required for various pipe sizes and locations, two general criteria must be met:

- The access hole or inlet structure must be large enough to accept the maximum pipe as shown in Table 13-5.
- Knowing the relative locations of any two pipes, compute:

$$K = \frac{R_1 + T_1 + R_2 + T_2 + 14\text{ in}}{\Delta} \tag{13.27}$$

where: R_1 and T_1 are interior radius and wall thickness of Pipe #1, in
R_2 and T_2 are interior radius and wall thickness of Pipe #2, in
Δ = angle between the pipes, degrees

Example Problem

Given: Pipe # 1 = 54 in
Pipe # 2 = 48 in
Δ = 140°

Solution: $$K = \frac{27\text{ in} + 5.5\text{ in} + 24\text{ in} + 5\text{ in} + 14\text{ in}}{140^\circ}$$
$$= 0.54\text{ in}/^\circ$$

Table 13-5 indicates that the minimum access hole barrel is 66 in. For the 66-in AH barrel, the Table indicates a maximum pipe size of 48 in. Because the maximum pipe size in the Example is 54 in, a 72-in access hole must be used.

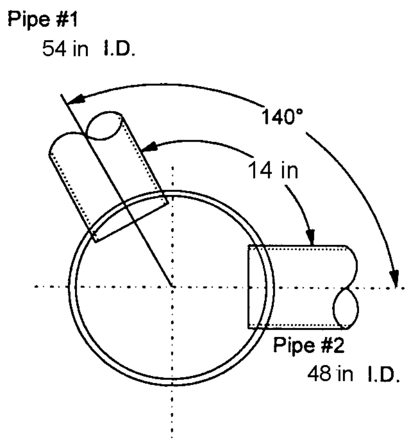


TABLE 13-5 — Access Hole Sizing

Manhole Dia. (in)	K (in/°)	Max Pipe Size (in)
27	0.24	15
42	0.37	27
48	0.42	30
54	0.47	36
60	0.52	42
66	0.58	48
72	0.63	54
78	0.68	60
84	0.73	66
90	0.79	72
96	0.84	72
102	0.89	78
108	0.94	84

For this Example, spacing is not critical and the pipe size governs. Had the Δ angle been 115° or less, the spacing would be critical, and a larger access hole barrel would have been required. If pipes are located at substantially different elevations, pipes may not conflict and the above analysis is unnecessary.

13.14 STORM DRAINS

13.14.1 Introduction

After the preliminary locations of inlets, connecting pipes and outfalls with tailwaters have been determined, the next logical step is the computation of the rate of discharge to be carried by each reach of the storm drain, and the determination of the size and gradient of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach, to the point where the storm drain connects with other drains or the outfall.

The rate of discharge at any point in the storm drain is not necessarily the sum of the inlet flow rates of all inlets above that section of storm drain. It is generally less than this total. The time of concentration is most influential and, as the time of concentration grows larger, the rainfall intensity to be used in the design grows smaller. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger using the shorter time even though the entire drainage area is not contributing. The prudent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment. See Section 13.6.2.3 for a discussion on time of concentration.

For ordinary conditions, storm drains should be sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's formula is recommended for capacity calculations. In locations such as depressed sections and underpasses where ponded water can be removed only through the storm drain system, a higher design frequency 50-yr should be considered to design the storm drain that drains the sag point. See Section 13.12.8 for a discussion on the location of flanking inlets. The main storm drain downstream of the depressed section should be designed by computing the hydraulic grade line and keeping the water surface elevations below the grates and/or established critical elevations for the design storm.

13.14.2 Design Procedures

The design of storm drainage systems is generally divided into the following operations:

Step 1 Determine inlet location and spacing as outlined earlier in this Chapter.

Step 2 Prepare the plan layout of the storm drainage system establishing the following design data:

a. location of storm drains;

b. direction of flow;

c. location of access holes; and

d. location of existing utilities (e.g., water, gas, underground cables and existing and proposed foundations).

Step 3 Determine drainage areas, runoff coefficients and a time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity. Calculate the discharge by (A)(C).

Step 4 Size the pipe to convey the discharge by varying the slope and pipe size as necessary. The storm drain systems are normally designed for full gravity-flow conditions using the design frequency discharges.

Step 5 Calculate travel time in the pipe to the next inlet or access hole by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and a new rainfall intensity at the next entry point.

Step 6 Calculate the new area (A) and multiply by the runoff coefficient (C), add to the previous (CA) and multiply by the new rainfall intensity to determine the new discharge. Determine the size of pipe and slope necessary to convey the discharge.

Step 7 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 8 Complete the design by calculating the hydraulic grade line as described in Section 13.15. The design procedure should include the following:

- Storm drain design computations can be made on forms as illustrated in Figure 13-17.

- All computations and design sheets should be clearly identified. The designer's initials and date of computations should be shown on every sheet. Voided or superseded sheets should be so marked. The origin of data used on one sheet but computed on another should be given.

13.14.3 Sag Point

As indicated above, the storm drain that drains a major sag point should be sized to accommodate the runoff from a 50-yr frequency rainfall. This can be done by actually computing the runby occurring at each inlet during a 50-yr rainfall and accumulating it at the sag point. The inlet at the sag point and the storm drain pipe leading from the sag point must be sized to accommodate this additional runby within the criteria established. See Section 13.9. To design the pipe leading from the sag point, it may be helpful to convert the additional runby created by the 50-yr rainfall into an equivalent CA that can be added to the design CA. This equivalent CA can be approximated by dividing the 50-yr runby by (I_{10}) in the pipe at the sag point.

Some designers may want to design separate systems to prevent the above-ground system from draining into the depressed area. This concept may be more costly but, in some cases, may be justified. Another method would be to design the upstream system for a 50-yr design to minimize the runby to the sag point. Each case must be evaluated on its own merits, and the impacts and risk of flooding a sag point assessed.

13.14.4 Hydraulic Capacity

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula, expressed by the following Equation:

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{13.28}$$

where:
- V = mean velocity of flow, ft/s
- n = Manning's roughness coefficient
- R = hydraulic radius, ft = area of flow divided by the wetted perimeter (A/WP)
- S = the slope of the energy grade line, ft/ft

In terms of discharge, the above formula becomes:

$$Q = VA = \frac{1.486}{n} AR^{2/3} S^{1/2} \tag{13.29}$$

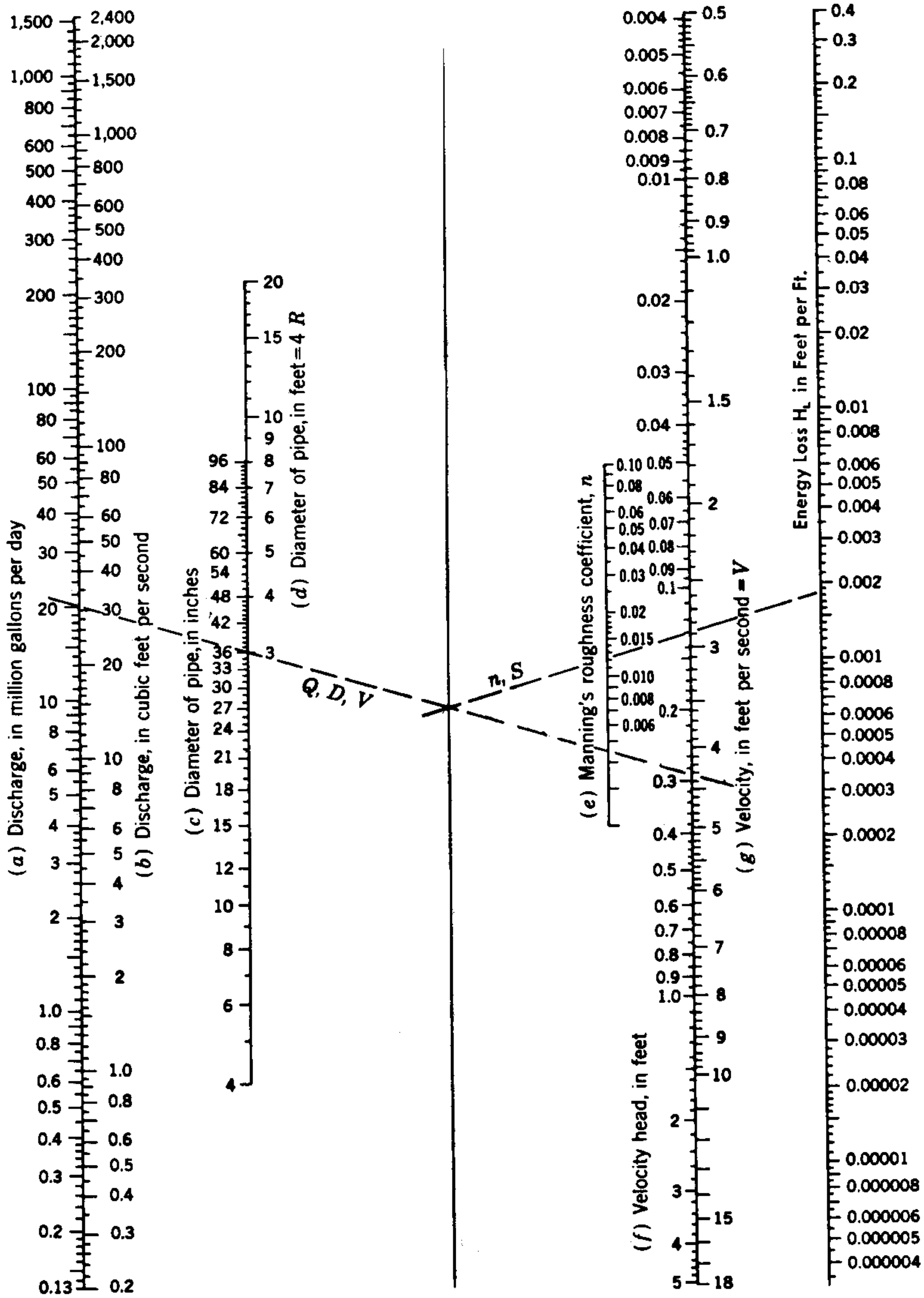
where:
- Q = rate of flow, ft^3/s
- A = cross sectional area of flow, ft^2

For circular storm drains flowing full, R = D/4 and Equations 13.28 and 13.29 become:

$$V = \frac{0.590}{n} D^{2/3} S^{1/2} \qquad Q = \frac{0.463}{n} D^{8/3} S^{1/2} \tag{13.30}$$

where:
- D = diameter of pipe, ft

The nomograph solution of Manning's formula for full flow in circular storm drains is shown on Figure 13-13, Figure 13-14 and Figure 13-15. Figure 13-16 has been provided to assist in the solution of Manning's Equation for partial full flow in storm drains.



Alignment chart for energy loss in pipes, for Manning's formula.
Note: Use chart for flow computations, $H_L = S$

Solution of Manning's Equation for Flow in Storm Drains - English Units
(Taken from "Modern Sewer Design" by American Iron and Steel Intitute)

FIGURE 13-13 — Manning's Formula for Full Flow in Storm Drains

Source: HEC 22 *(9)*.

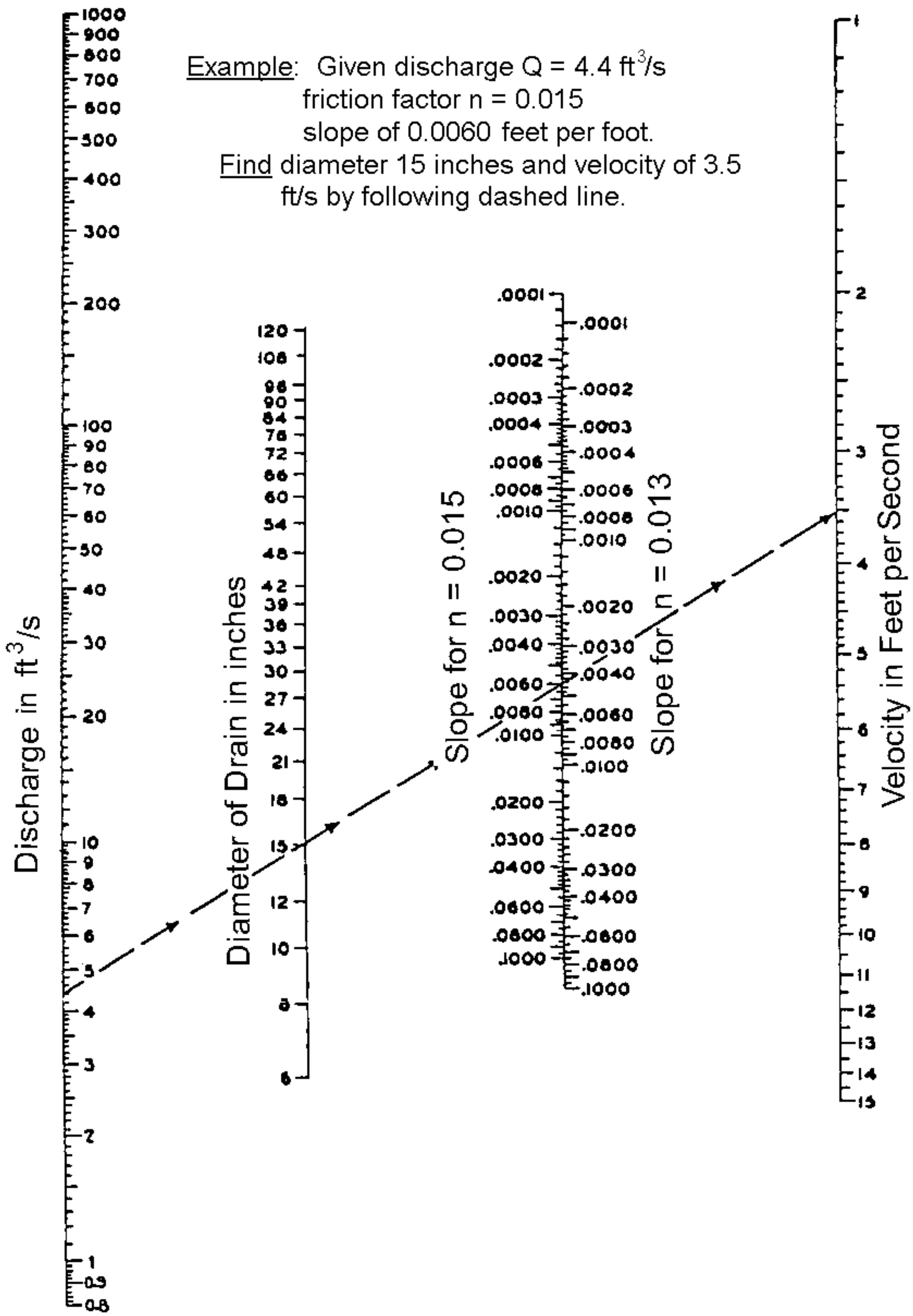


FIGURE 13-14 — Nomograph for Computing Required Size of Circular Drain for Full Flow (n = 0.013 or 0.015)

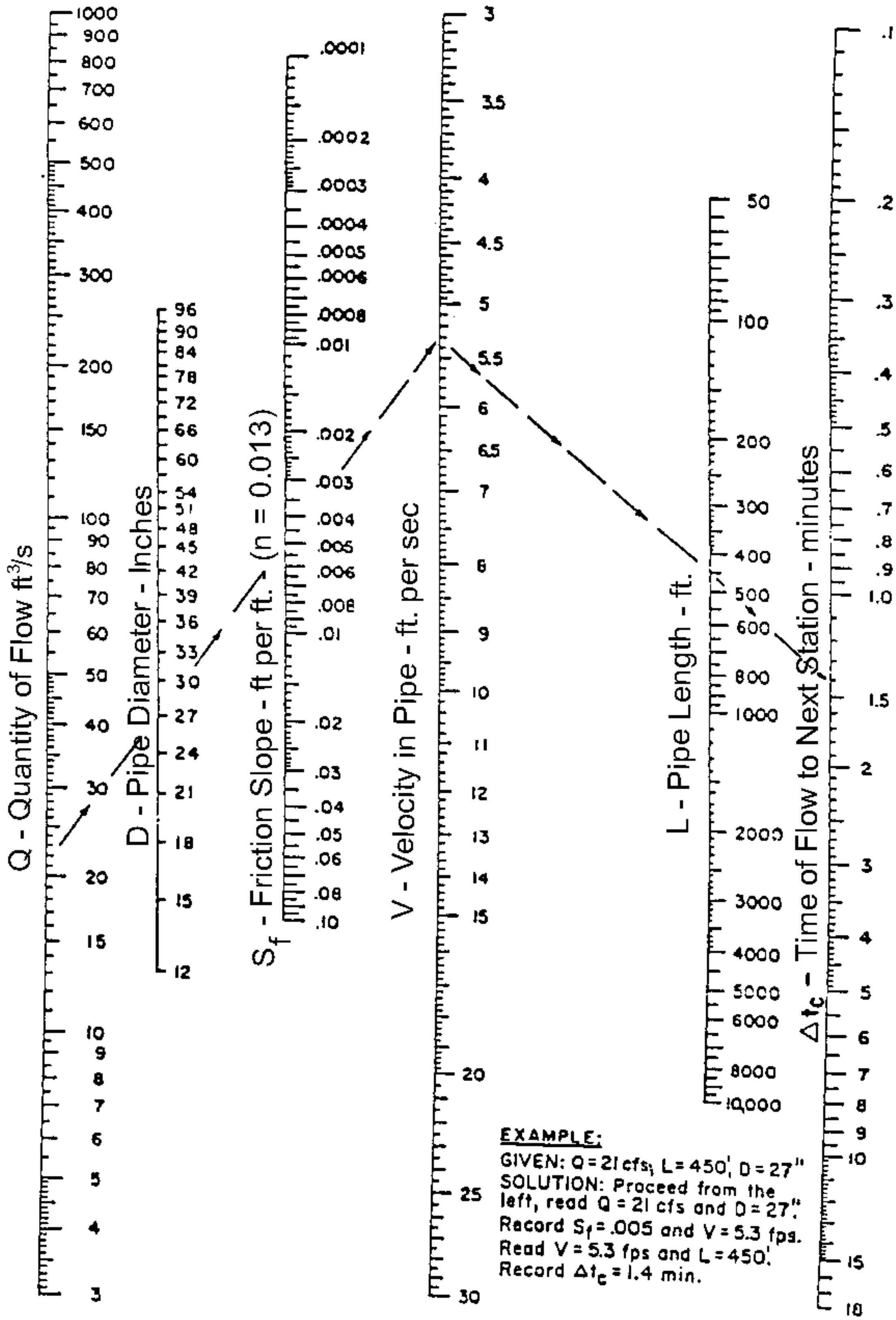
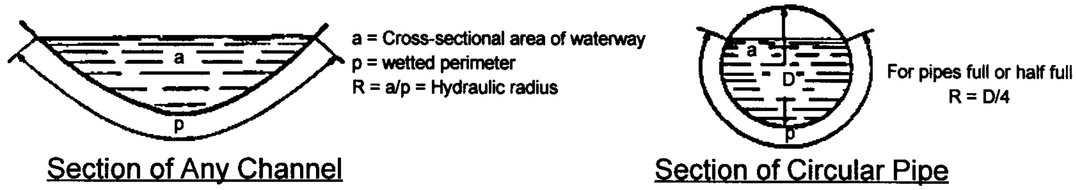


FIGURE 13-15 — Concrete Pipe Flow Nomograph



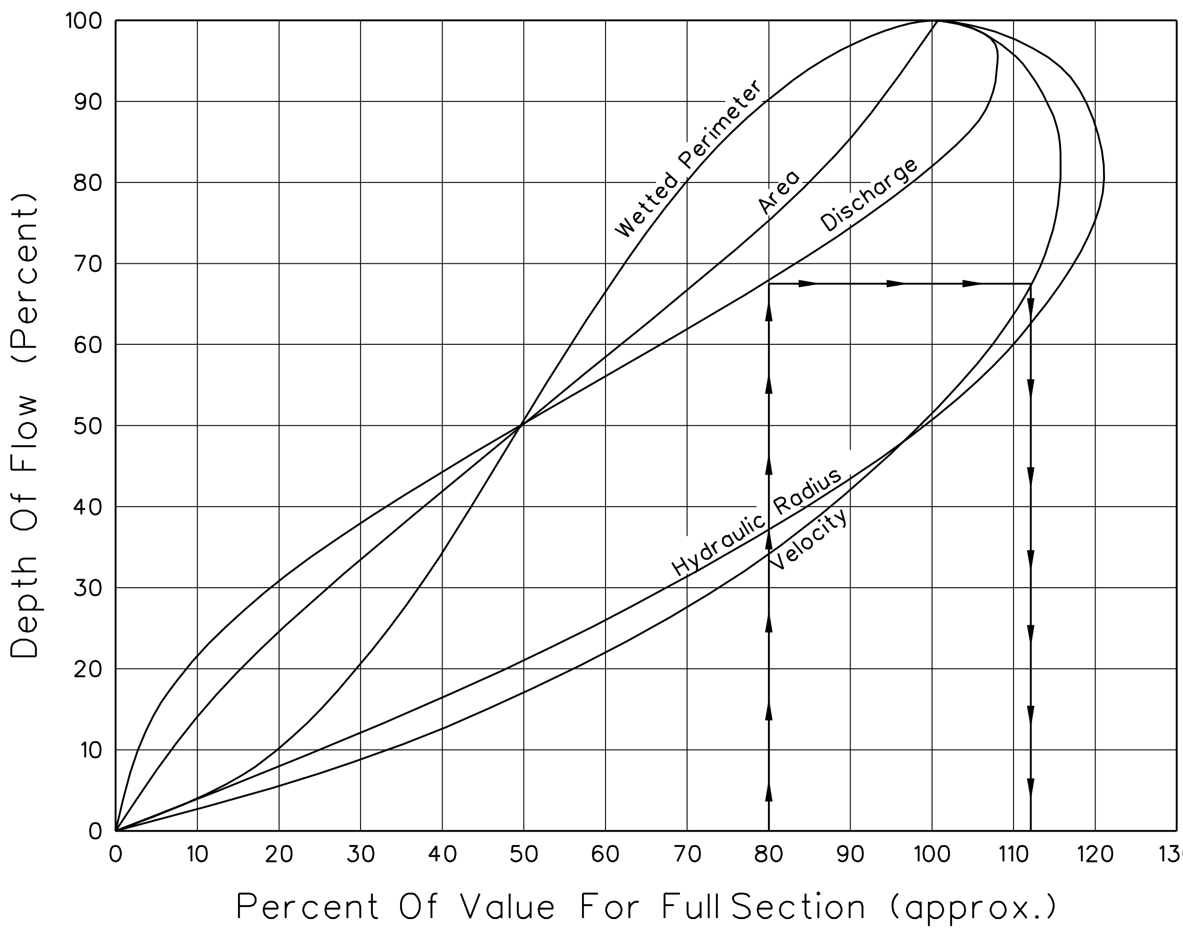
V = Average or mean velocity in ft/s

Q = a V = Discharge of pipe or channel in ft^3/s

n = Coefficient of roughness of pipe or channel surface

S = Slope of hydraulic gradient (water surface in open channels or pipes not under pressure, same as slope of channel or pipe invert only when flow is uniform in constant section)

Hydraulic Elements of Channel Sections



Source: Reference *(3)*.

FIGURE 13-16 — Values of Hydraulic Elements of Circular Section for Various Depths of Flow

13.14.5 Minimum Grades

All storm drains should be designed such that velocities of flow will not be less than 2 ft/s at design flow. For very flat grades, the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to ensure that there is sufficient velocity in all drains to deter settling of particles. Minimum slopes required for a velocity of 3 ft/s can be calculated by the Manning's formula, or use values given in Table 13-6:

$$S = \left[\frac{nV}{1.486\ R^{2/3}} \right]^2 \qquad (13.31)$$

13.14.6 Curved Alignment

Curved storm drains are permitted where necessary. Long-radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 48 in and larger. Short-radius bend sections are also available and can be used if there is not room for the long-radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. Using large access holes solely for changing direction may not be cost effective on large-size storm drains.

TABLE 13-6 — Minimum Slopes Necessary To Ensure 3 ft/s In Storm Drains Flowing Full

Pipe Size (in)	Full Pipe (ft^3/s)	Minimum Slopes (ft/ft)		
		n = 0.012	n = 0.013	n = 0.024
8	1.05	0.0064	0.0075	0.0256
10	1.64	0.0048	0.0056	0.0190
12	2.36	0.0037	0.0044	0.0149
15	3.68	0.0028	0.0032	0.0111
18	5.30	0.0022	0.0026	0.0087
21	7.22	0.0018	0.0021	0.0071
24	9.43	0.0015	0.0017	0.0059
27	11.93	0.0013	0.0015	0.0051
30	14.73	0.0011	0.0013	0.0044
33	17.82	0.00097	0.0011	0.0039
36	21.21	0.00086	0.0010	0.0034
42	28.86	0.00070	0.00082	0.0028
48	37.70	0.00059	0.00069	0.0023
54	47.71	0.00050	0.00059	0.0020
60	58.90	0.00044	0.00051	0.0017
66	71.27	0.00038	0.00045	0.0015
72	84.82	0.00024	0.00040	0.0014

Storm Drain Computation Sheet

Computed _______ Date _______

Checked _______ Date _______

Route ______________

Section ______________

County ______________

Station		Length (ft)	Drainage Area A (ac)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (in/h)	Total Runoff 0.0278 C I A = Q (ft^3/s)	Diameter Pipe (in)	Capacity Full (ft^3/s)	Velocity (ft/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (ft/ft)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

FIGURE 13-17 — Storm Drain Computation Sheet

13.15 HYDRAULIC GRADE LINE

13.15.1 Introduction

The hydraulic grade line (HGL) is the last important feature to be established for the hydraulic design of storm drains. This grade line aids the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating from a flood of design frequency.

In general, if the HGL is above the crown of the pipe, pressure-flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. A special concern with storm drains designed to operate under pressure-flow conditions is that inlet surcharging and possible access hole lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drain systems can often alternate between pressure and open channel flow conditions from one section to another.

The detailed methodology employed in calculating the HGL through the system begins at the system outfall with the tailwater elevation. If the outfall is an existing storm drain system, the HGL calculation must begin at the outlet end of the existing system and proceed upstream through this in-place system, then upstream through the proposed system to the upstream inlet. The same considerations apply to the outlet of a storm drain as to the outlet of a culvert. See Figure 9-4 for a sketch of a culvert outlet that depicts the difference between the HGL and the energy grade line (EGL). Usually, it is helpful to compute the EGL first, then subtract the velocity head ($V^2/2g$) to obtain the HGL.

Water quality treatment facilities placed in the storm drain system may significantly reduce the system capacity. The computation of the hydraulic grade line shall include the head loss caused by the treatment facility to determine if the facility causes an intolerable reduction in system capacity. If the hydraulic grade line upstream of the treatment facility is excessive, the treatment facility may be lowered in some cases to reduce the hydraulic grade line upstream of the device. If the device causes excessive head loss and cannot be lowered to reduce the upstream hydraulic grade line, then an alternative device having a lower head loss shall be considered.

13.15.2 Tailwater

For most design applications, the tailwater will either be above the crown of the outlet or can be considered to be between the crown and critical depth. To determine the EGL, begin with the tailwater elevation or $(d_c + D)/2$ (use only as an approximation for hand calculations; it is not valid if the designer is determining friction losses by computing backwater through the pipe), whichever is higher, add the velocity head for full flow and proceed upstream to compute all losses (e.g., exit losses, friction losses, junction losses, bend losses, entrance losses) as appropriate.

An exception to the above might be an outfall with low tailwater when a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full-flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the tailwater, whichever is higher.

When estimating tailwater depth on the receiving stream, the prudent designer will consider the joint or coincidental probability of two events occurring at the same time. For a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving stream. A short-duration storm, which causes peak discharges on a small basin, may not be critical for a larger basin. Also, it may safely be assumed that, if the same storm causes peak discharges on both basins, the peaks will be out of phase. When applying this approach, it is necessary to perform two independent sets of hydraulic grade line computations for the combinations of frequencies. An example of such an analysis is shown in Table 13-7.

The hydraulic grade line using a 100-yr flood stage of the receiving waters should be checked to determine if reverse flow conditions may occur. If reverse flow conditions are possible, then flap gates and/or a shut-off gate may be considered.

13.15.3 Exit Loss

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion, such as an endwall, the exit loss is:

$$H_o = C_o \left[\frac{V^2}{2g} - \frac{V_d^2}{2g} \right] \tag{13.32}$$

where: V = average outlet velocity, ft/s
V_d = channel velocity downstream of outlet, ft/s
C_o = exit loss coefficient 0.3

Note that, when $V_d = 0$ as in a reservoir, the exit loss is one velocity head. For partial full flow where the pipe outlets into a channel with moving water, the exit loss may be reduced to virtually zero.

TABLE 13-7 — Example of Joint Probability Analysis for Streams near Virginia Beach, VA (early 1970s)

Area Ratio	Frequencies for Coincidental Occurrence			
	10-Yr Design		100-Yr Design	
	Main Stream	Tributary	Main Stream	Tributary
10 000 to 1	1 10	10 1	2 100	100 2
1000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

13.15.4 Bend Loss

The bend loss coefficient for storm drain design is minor but can be evaluated using the formula:

$$h_b = 0.0033\,(\Delta)(V_o^2\,/\,2g) \tag{13.33}$$

where: Δ = angle of curvature, degrees

13.15.5 Pipe Friction Losses

The friction slope is the energy gradient in ft/ft for that run. The friction loss is simply the energy gradient multiplied by the length of the run in feet. Energy losses from pipe friction may be determined by rewriting Manning's Equation with terms as previously defined:

$$S_f = [Qn/1.486\ AR^{2/3}]^2 \tag{13.34}$$

The head losses due to friction may be determined by the formula:

$$H_f = S_f L \tag{13.35}$$

Manning's Equation can also be written to determine friction losses for storm drains as follows:

$$H_f = 2.87\ n^2 V^2 L / D^{4/3} \tag{13.36}$$

$$H_f = \frac{64.4\ n^2\ L}{R^{4/3}} \left(\frac{V^2}{2g} \right) \tag{13.37}$$

where:
- H_f = total head loss due to friction, ft
- n = Manning's roughness coefficient
- D = diameter of pipe, ft
- L = length of pipe, ft
- V = mean velocity, ft/s
- R = hydraulic radius, ft
- g = 32.2 ft/s^2
- S_f = slope of hydraulic grade line, ft/ft

13.15.6 Access Hole Losses

The head loss encountered from one pipe to another through an access hole is commonly represented as being proportional to the velocity head at the outlet pipe. Using K to signify this constant of proportionality, the energy loss is approximated as (K)(V_o^2/2g). Experimental studies have determined that the K value can be approximated as follows:

$$K = K_o C_D C_d C_Q C_p C_B \tag{13.38}$$

where:
- K = adjusted loss coefficient
- K_o = initial head loss coefficient based on relative access hole size
- C_D = correction factor for pipe diameter (pressure flow only)
- C_d = correction factor for flow depth (non-pressure flow only)
- C_Q = correction factor for relative flow
- C_p = correction factor for plunging flow
- C_B = correction factor for benching

13.15.6.1 Relative Access Hole Size

K_o is estimated as a function of the relative access hole size and the angle of deflection between the inflow and outflow pipes. See Figure 13-18:

$$K_o = 0.1(b/D_o)(1 - \sin\theta) + 1.4(b/D_o)^{0.15}\sin\theta \tag{13.39}$$

where: θ = the angle between the inflow and outflow pipes, degrees
b = access hole diameter, in
D_o = outlet pipe diameter, in

13.15.6.2 Pipe Diameter

A change in head loss due to differences in pipe diameter is only significant in pressure-flow situations where the depth in the access hole to outlet pipe diameter ratio, d/D_o, is greater than 3.2. Therefore, it is only applied in such cases:

$$C_D = (D_o / D_i)^3 \tag{13.40}$$

where: D_i = incoming pipe diameter, in
D_o = outgoing pipe diameter, in

13.15.6.3 Flow Depth

The correction factor for flow depth is significant only in free surface flow or low pressures, where the d/D_o ratio is less than 3.2, and is only applied in such cases. Water depth in the access hole is approximated as the level of the hydraulic grade line at the upstream end of the outlet pipe. The correction factor for flow depth, C_d, is calculated by the following:

FIGURE 13-18 — Deflection Angle

where: d = water depth in access hole above outlet pipe, ft
D_o = outlet pipe diameter, ft

13.15.6.4 Relative Flow

The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow and the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

$$C_Q = (1 - 2\sin\theta)\left(1 - \frac{Q_i}{Q_o}\right)^{0.75} + 1 \qquad (13.42)$$

where:
- C_Q = correction factor for relative flow
- θ = the angle between the inflow and outflow pipes, degrees
- Q_i = flow in the inflow pipe, ft^3/s
- Q_o = flow in the outlet pipe, ft^3/s

As can be seen from the Equation, C_Q is a function of the angle of the incoming flow and the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the access hole shown in Figure 13-19 and assume the following two Cases to determine the impact of Pipe 2 entering the access hole:

$$C_{Q3\text{-}1} = (1 - 2\sin 180°)(1 - 3.2/4.2)^{0.75} + 1 = 1.34$$

<u>Case 1</u>

Q_1 = 3.2 ft^3/s, Q_2 = 1.0 ft^3/s
Q_3 = 4.2 ft^3/s, then C_Q = 1.34

<u>Case 2</u>

Q_1 = 1.0 ft^3/s, Q_2 = 3.2 ft^3/s
Q_3 = 4.2 ft^3/s, then C_Q = 1.76

FIGURE 13-19 — Relative Flow Effect

13.15.6.5 Plunging Flow

The correction factor for plunging flow, C_p, is calculated by the following:

$$C_p = 1 + 0.2\left[\frac{h}{D_o}\right]\left[\frac{(h-d)}{D_o}\right] \qquad (13.43)$$

where:
- C_p = correction for plunging flow
- h = vertical distance of plunging flow from flow line of incoming pipe to the center of outlet pipe, ft
- D_o = outlet pipe diameter, ft
- d = water depth in access hole, ft

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the access hole, on the inflow pipe for which the head loss is being calculated. Using the notations in Figure 13-19 for the Example, C_p is calculated for Pipe # 1 when Pipe # 2 discharges plunging flow. The correction factor is only applied when h > d.

13.15.6.6 Benching

The correction for benching in the access hole, C_B, is obtained from Table 13-8. Benching tends to direct flows through the access hole, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

13.15.6.7 Summary

In summary, to estimate the head loss through an access hole from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the outflow pipe to estimate the minor loss for the connection.

13.15.7 <u>Hydraulic Grade Line Design Procedure</u>

The equations and charts necessary to manually calculate the location of the hydraulic grade line are included in this Chapter. A step-by-step procedure is given to manually compute the HGL. Table 13-9 can be used to document the procedure.

TABLE 13-8 — Correction for Benching

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02

*pressure flow, $d/D_o > 3.2$
**free surface flow, $d/D_o < 1.0$

Schematic Representation of Benching Types

If the HGL is above the pipe crown at the next upstream access hole, pressure-flow calculations are indicated; if it is below the pipe crown, then open channel flow calculations should be used at the upstream access hole. The procedures outlined here assume full flow. The process is

TABLE 13-9 — Hydraulic Grade Line Computation Form

Station	TW	D_o	Q_o	L_o	V_o	$V_o^2/2g$	H_o	SF_o	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_0 (2)+(7)	EGL_i (10)+(18) +(19)	HGL (EGL) – (7)	TOC Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

repeated throughout the storm drain system. If all HGL elevations are acceptable, then the hydraulic design is adequate. If the HGL exceeds an inlet elevation, then adjustments to the trial design must be made to lower the water surface elevation.

See Figure 13-20 for a sketch depicting the use of energy losses in developing a storm drain system:

Step 1 Enter in Column 1 the station for the junction immediately upstream of the outflow pipe. HGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Column 2 the tailwater elevation if the outlet will be submerged during the design storm; otherwise, refer to the tailwater discussion in Section 13.15.2 for procedure.

Step 3 Enter in Column 3 the diameter, D_o, of the outflow pipe.

Step 4 Enter in Column 4 the design discharge, Q_o, for the outflow pipe.

Step 5 Enter in Column 5 the length, L_o, of the outflow pipe.

Step 6 Enter in Column 6 the outlet velocity of flow, V_o.

Step 7 Enter in Column 7 the velocity head, $V_o^2/2g$.

Step 8 Enter in Column 8 the exit loss, H_o.

Step 9 Enter in Column 9 the friction slope (SF_o) in ft/ft of the outflow pipe. This can be determined by using Equation 13.34. *Note: Assumes full-flow conditions.*

Step 10 Enter in Column 10 the friction loss (H_f) that is computed by multiplying the length (L_o) in Column 5 by the friction slope (SF_o) in Column 9. On curved alignments, calculate curve losses by using the formula $H_c = 0.0033\ (\Delta)(V_o^2/2g)$, where Δ = angle of curvature in degrees, and add to the friction loss.

Step 11 Enter in Column 11 the initial head loss coefficient, K_o, based on relative access hole size as computed by Equation 13.39.

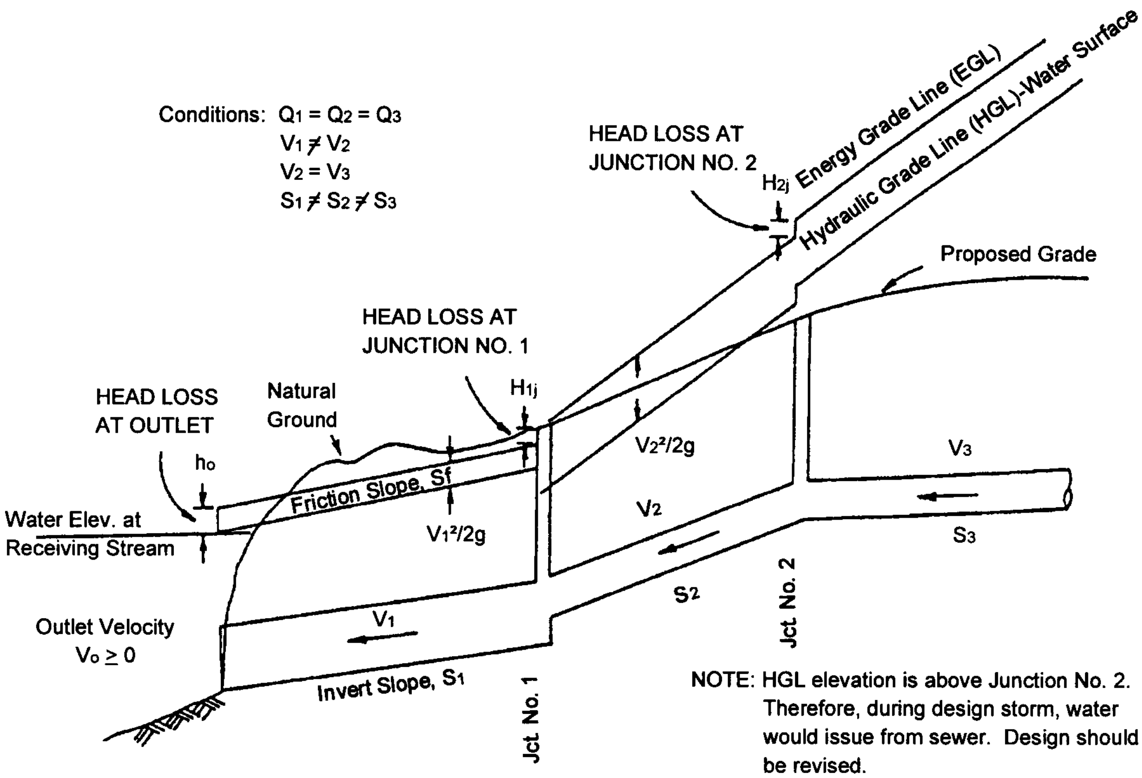
Step 12 Enter in Column 12 the correction factor for pipe diameter, C_D, as computed by Equation 13.40.

Step 13 Enter in Column 13 the correction factor for flow depth, C_d, as computed by Equation 13.41. *Note: This factor is only significant where the d/D_o ratio is less than 3.2.*

Step 14 Enter in Column 14 the correction factor for relative flow, C_Q, as computed by Equation 13.42.

Step 15 Enter in Column 15 the correction factor for plunging flow, C_p, as computed by Equation 13.43. *Note: This correction factor is only applied when $h > d$.*

IMPROPER DESIGN



PROPER DESIGN

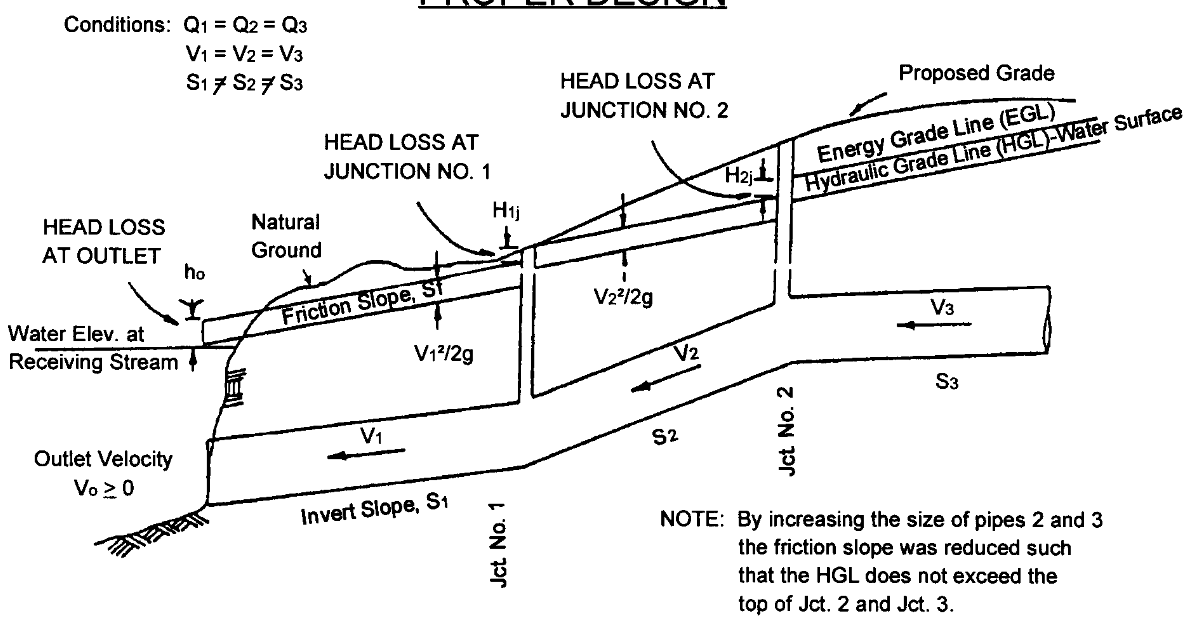


FIGURE 13-20 — Use of Energy Losses in Developing a Storm Drain System

Step 16 Enter in Column 16 the correction factor for benching, C_B, as determined in Table 13-8.

Step 17 Enter in Column 17 the value of K as computed by Equation 13.38.

Step 18 Enter in Column 18 the value of the total access hole loss, $KV_o^2/2g$.

Step 19 If the tailwater submerges the outlet end of the pipe, enter in Column 19 the sum of Column 2 (TW elevation) and Column 7 (exit loss) to get the EGL at the outlet end of the pipe. If the pipe is flowing full, but the tailwater is low, the EGL will be determined by adding the velocity head to $(d_c + D)/2$.

Step 20 Enter in Column 20 the sum of the friction head (Column 10), the access hole losses (Column 18), and the energy grade line (Column 19) at the outlet to obtain the EGL at the inlet end. This value becomes the EGL for the downstream end of the upstream pipe.

Step 21 Determine the HGL (Column 21) throughout the system by subtracting the velocity head (Column 7) from the EGL (Column 20).

Step 22 Check to make certain that the HGL is below the level of allowable highwater at that point. If the HGL is above the finished grade elevation, water will exit the system at this point for the design flow. *Note: TOC is top of curb.*

Calculating backwater for partial full-flow conditions through the pipe is tedious and may require many iterations to determine if gravity-flow or pressure-flow conditions exist. The junction losses may be quite different depending on flow types. Momentum methods may be required to estimate hydraulic jump conditions. The above procedure is simplistic and may not apply well to gravity-flow conditions, especially if supercritical flow is encountered.

The above procedure applies to pipes that are flowing full, as should be the condition for the design of new systems. If a partial full-flow condition exists, the EGL is located one velocity head above the water surface.

13.16 WATER QUALITY TREATMENT

Many storm drain systems are required to be fitted or retrofitted with stormwater treatment facilities. The treatment facilities may remove oils, grease, floatables and sediment. The amount of pollutants that treatment facilities can remove may vary depending on the type of facility installed. Commercially available facilities may provide design data for use in selecting the size and type needed. Chapter 15 also provides some additional aspects of storm water quality relating to storm drains. Various treatment facility types may not provide an “equivalent” amount of pollutant removal or level of service. The criteria for pollutant removal must be established before the alternative facilities are designed to provide for alternative bidding. Substitutions of other treatment systems must be designed to meet the design criteria used for those alternatives shown in the contract plans.

13.17 INVERTED SIPHONS

An inverted siphon carries the flow under an obstruction such as sanitary sewers, water mains or any other structure or utility lane that is in the path of the storm drain line. The storm drain invert is lowered at the obstacle and is raised again after the crossing. A minimum of two barrels with 3 ft/s velocity is recommended. The inlet and outlet structures should be designed by keeping the normal flow in one barrel to provide the required minimum velocity for self-cleaning and servicing. The criteria for designing inverted siphons can be found in most hydraulics textbooks.

See the Culverts Chapter, Appendix 9.D, for an inverted siphon design example.

13.18 UNDERDRAINS

In certain areas, groundwater can be a significant problem because it attacks foundations, substructures, subgrades and other aspects of highway components. In most soils where groundwater is a problem, a system of underdrains, installed for the removal of excess moisture, can be a very useful feature in the overall roadway design. Underdrains may take the form of networks of perforated (or otherwise permeable) pipe, French drains or collector fields. Where such appurtenances are needed, the additional expense in their installation is usually fully justified in terms of future savings in roadway and structure maintenance costs.

Percolation rates for groundwater may be obtained from NRCS offices, measured or simply estimated. Collector pipe sizes and networks may then be established for the removal of that water. French drains can be very useful where the unwanted groundwater percolation rates are relatively high. Collector fields may be useful where reasonable outfalls for groundwater are not available. All of the above appurtenances may be enhanced by the use of some type of geotextile filter material.

13.19 COMPUTER PROGRAMS

The computer market offers several reliable software packages to aid the storm drain design. UDOT currently uses Storm and Sanitary Sewer®, part of the Bently SelectCADD® modules. This software is very useful to the designers since it uses the design information from the plans. Other reliable software is StormCADD® and FlowMaster® from Haested Methods.

Storm drainage software has a wide range of capabilities. The selection of software may be based on the various features that the software provides. The hydrologic methods are of primary importance. In addition to the Rational method, some software provides various hydrographic techniques such as NRCS, Kinematic Wave, Colorado Urban Hydrograph (CHUP) and Santa Barbara Urban Hydrograph (SBUH). The hydraulic capabilities of storm drain software may include detailed inlet capture modeling, junction loss and hydraulic grade line analysis. Some programs have design capabilities to optimize inlet spacing or pipe sizes. The quality of the analysis provided by the software may vary. The user must be aware of the limitations of the software and must carefully review the results to determine if the methods are properly applied and if the results are reasonable. Many software packages are interfaced with GIS systems. Some software packages will provide details of the storm drain suitable for construction plans, and some software packages are integrated with the roadway design software packages.

A simple spreadsheet program, is a very useful tool for designing a storm drainage system. The information on a spreadsheet is very accessible and equations can be modified to fit the needs of the designer.

13.20 REFERENCES

(1) AASHTO, *A Policy on Geometric Design of Highways and Streets*, Task Force on Geometric Design, 2001.

(2) AASHTO, *Highway Drainage Guidelines*, Chapter 9, “Storm Drain Systems,” Task Force on Hydrology and Hydraulics, 2003.

(3) Chow, V.T., *Open Channel Hydraulics*, New York, McGraw-Hill Book Company, 1959.

(4) Federal Highway Administration, *Bridge Deck Drainage Systems*, Hydraulic Engineering Circular No. 21, FHWA-SA-92-010, 1993.

(5) Federal Highway Administration, *Drainage of Highway Pavements*, Hydraulic Engineering Circular No. 12, FHWA-TS-84-202, 1984.

(6) Federal Highway Administration, *HYDRAIN, Drainage Design Computer System*, Version 6.1, FHWA-IF-99-008, 1999.

(7) Federal Highway Administration, Hydraulic Design Series No. 3, *Design Charts for Open-Channel Flow*, FHWA-EPD-86-102, 1961.

(8) Federal Highway Administration, *Pavement and Geometric Design Criteria For Minimizing Hydroplaning*, FHWA-RD-79-31, December 1979.

(9) Federal Highway Administration, *Urban Drainage Design Manual*, Hydraulic Engineering Circular No. 22, FHWA-SA-96-078, 2001.

(10) NCHRP Project 1-29, *Improved Surface Drainage of Pavements*, Final Report, Pennsylvania Transportation Institute, July 1998.

(11) NCHRP Research Results Digest, Number 243, “Proposed Design Guidelines for Reducing Hydroplaning on New and Rehabilitated Pavements,” September 1999.